PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXIX—No. 1

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January, 1913

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(INSTITUTED 1852)

VOL. XXXIX—No. 1

JANUARY, 1913

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1913

American Society of Civil Engineers

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CHARLES D. MARX

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TO INVESTIGATE CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS: Alfred Noble, S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, John A. Bensel.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER....................5913 Columbus.
CABLE ADDRESS......................"Ceas, New York."

Vol. XXXIX. JANUARY, 1913. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS

OF THE SOCIETY

December 18th, 1912.—The meeting was called to order at 8.30 P. M.; Director T. Kennard Thomson in the chair; Chas. Warren Hunt, Secretary; and present, also, 83 members and 13 guests.

A paper by Spencer Miller, M. Am. Soc. C. E., entitled "Prevention of Mosquito Breeding," was presented by the author and illustrated with lantern slides. The Secretary read communications on the subject from Messrs. Harold Farnsworth Gray, and J. J. Rosenthal, and the paper was further discussed by Dr. Ralph H. Hunt and Messrs. Robert A. Rutherfurd, James Owen, Kenneth Allen, George A. Flynn, Frederic A. Snyder, George N. Cole, and the author.

A paper entitled "The Sanitation of Construction Camps," by Harold Farnsworth Gray, Jun. Am. Soc. C. E., was presented by title. A written discussion on the subject by R. C. Hardman, Assoc. M.

Am. Soc. C. E., was read by the Secretary, and the paper was discussed orally by Messrs. Frank E. Winsor, George A. Flynn, and Spencer Miller.

The Secretary announced the following deaths:

Alfred Pancoast Boller (*Vice-President*), elected Member, December 4th, 1867; died December 9th, 1912.

Charles Albert Allen, elected Member, June 4th, 1879; died December 9th, 1912.

Thomas Chalkley James Baily, Jr., elected Member, October 4th, 1905; died December 7th, 1912.

Robert Ballard, elected Member, September 1st, 1880; died November 22d, 1912.

Andrew Bell, elected Member, September 5th, 1883; died October 23d, 1912.

James Moore Shanly, elected Member, July 6th, 1887; died November 28th, 1912.

Henry England Grimm, elected Junior, October 2d, 1900; Associate Member, November 4th, 1908; died December 12th, 1912.

James Breckinridge Speed, elected Associate, May 2d, 1888; died July 7th, 1912.

Adjourned.

January 8th, 1913.—The meeting was called to order at 8.30 p. m.; President Ockerson in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 84 members and 12 guests.

The minutes of the meeting of November 20th and December 4th, 1912, were approved as printed in *Proceedings* for December, 1912.

A paper by H. T. Cory, M. Am. Soc. C. E., entitled "Irrigation and River Control in the Colorado River Delta," was presented by the Assistant Secretary, who also read written communications on the subject from Messrs. L. J. Le Conte and Morris Knowles.

The Assistant Secretary announced the election of the following candidates on January 7th, 1913:

As Members

Ralph Budd, Portland, Ore.
William Stoddert Caruthers, Sacramento, Cal.
Elmer Ellsworth Colby, Alva, Okla.
William McClurg Donley, Pittsburgh, Pa.
Johannes Marcelius Hammer, Pittsburgh, Pa.
Frank Willard Hanna, Washington, D. C.
Washington J Miller, San Francisco, Cal.
Jerome Newman, San Francisco, Cal.
Royal Albert Polhamus, Chicago, Ill.

As Associate Members

Amon Benjamin Brown, Rupert, Idaho
Levant R Brown, Lafayette, Ind.
Charles Stephen Christian, Texarkana, Ark.
Stafford Xavier Comber, New York City
Warren Hoover Converse, Jr., Chattanooga, Tenn.
Sidney Wooddell Cooper, Roswell, N. Mex.
Thomas Meredith Davidson, Chicago, Ill.
Harry Johnson Deutschbein, Little Hocking, Ohio
Charles Slauter Doron, Brooklyn, N. Y.
Octavio Manuel Figueroa, Buenos Aires, Argentine Republic
Joseph Gallagher, Manila, Philippine Islands
John Huston Clark Gregg, New Paltz, N. Y.
Charles Marsh Husband, Bellevue, Pa.
Amund Marius Korsmo, Oakley, Idaho
William Eustace Maclean, Vancouver, B. C., Canada
Thomas Rucker McSwain, Tulare, Cal.
William Benjamin Moss, Tompkinsville, N. Y.
George Nelson, Los Angeles, Cal.
Howard Eastwood Phelps, Boulder, Colo.
Percy Allen Seibert, La Paz, Bolivia
Plumer Henry Smith, Texas City, Tex.
John Lynch Stanage, Fort Worth, Tex.
Harry Stewart Van Scoyoc, Montreal, Que., Canada
Vernon Gregg Watters, Live Oak, Fla.
Charles Asa Dilts Young, Seattle, Wash.

As Associates

Frederick Calvin Davis, San Francisco, Cal.
Ralph Leroy Parshall, Fort Collins, Colo.

As Juniors

Paul Bailey, Sacramento, Cal.
George Lindsley Burr, New York City
Cecil Ward Howard, Cincinnati, Ohio
Sydney Bishop Lamb, Laramie, Wyo.
William Bruce McMillan, San José, Cal.
Stuart Fabian Magor, Torrance, Cal.
Henry Brackette Parker, Albany, N. Y.
Edmund John Pickford, Johannesburg, South Africa
Albert Fredrick Porzelius, Little Rock, Ark.
Walter Mickle Smith, Jr., West New Brighton, N. Y.
Alfred Kenneth Starkweather, Bloomfield, N. J.
James Bertrand Wells, Palo Alto, Cal.
Frederick John Wright, Paterson, N. J.
George Leland Youmans, Winesap, Wash.

The Assistant Secretary announced the transfer of the following candidates on January 7th, 1913:

FROM ASSOCIATE MEMBER TO MEMBER

JOHN SEBASTIAN CONWAY, Washington, D. C.
JOSEPH SPENCER CRANE, Newark, N. J.
JOHN WILLIAMS DOTY, New York City
HOWELL TRACY FISHER, Montreal, Que., Canada
WILLIAM KENDRICK HATT, Lafayette, Ind.
WILLIAM AUGUST HUNICKE, Homer, La.
CLYDE WEBSTER MACCORNACK, Phœnixville, Pa.
HOLTON DUNCAN ROBINSON, New York City
CHARLES PERKINS WEBBER, Tierra Blanca, Ver., Mexico
WILLIAM HENRY YATES, Albany, N. Y.

FROM JUNIOR TO ASSOCIATE MEMBER

ROGER DERBY BLACK, Albany, N. Y.
MYRON CARLOS BURR, Fort George, B. C., Canada
WAYNE JOSEPH BURTON, Kansas City, Mo.
FOSTER BALDWIN CROCKER, Rome, N. Y.
GAGE HASELTON, Portland, Ore.
FRANCIS WILLIAM MCKINNEY, Baltimore, Md.
HUNTER IMBODEN SNYDER, Jacksonville, Fla.

FROM JUNIOR TO ASSOCIATE

EDWARD JOHN MEHREN, New York City

The Assistant Secretary announced the following deaths:

MORRIS M. DEFREES, elected Member, March 3d, 1880; died October 16th, 1912.

JULIAN THORNLEY, elected Associate Member, January 2d, 1901; Member, September 5th, 1905; died December 21st, 1912.

JOHN HAWKESWORTH, elected Junior, September 6th, 1904; Associate Member, November 4th, 1908; died December 10th, 1912.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

February 5th, 1913.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "Characteristics of Cup and Screw Current Meters; Performance of These Meters in Tail-Races and Large Mountain Streams; Statistical Synthesis of Discharge Curves," by B. F. Groat, Assoc. M. Am. Soc. C. E.; and "The Infiltration of Ground-Water into Sewers," by John N. Brooks, Jun. Am. Soc. C. E.

These papers were printed in *Proceedings* for December, 1912.

February 19th, 1913.—8.30 P. M.—Two papers will be presented for discussion at this meeting, as follows: "A Suggested Improvement in Building Water-Bound Macadam Roads," by J. L. Meem, Assoc. M. Am. Soc. C. E.; and "On Long-Time Tests of Portland Cement," by I. Hiroi, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for December, 1912.

March 5th, 1913.—8.30 P. M.—A regular business meeting will be held, and a paper by Caleb Mills Saville, M. Am. Soc. C. E., entitled "Hydrology of the Panama Canal," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

March 19th, 1913.—8.30 P. M.—At this meeting a paper by E. J. Schneider, M. Am. Soc. C. E., entitled "Construction Problems, Dumbarton Bridge, Central California Railway," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be

performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and, on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Gavin N. Houston, M. Am. Soc. C. E., 409 Equitable Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, and, until further notice, will take place at the Colorado Traffic Club.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association

On March 14th, 1912, the Atlanta Association of Members of the American Society of Civil Engineers was organized, with the following officers: Arthur Pew, President; William A. Hansell, Jr., Secretary; and Messrs. James N. Hazlehurst and Alexander Bonnyman, Members of the Executive Committee. The Association will hold its meetings in the house of the University Club.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 302 Board of Trade Building, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18-1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31st, 1912.

In compliance with the Constitution, the Board of Direction presents its report for the year ending December 31st, 1912.

The Society is growing very rapidly, and in this report, in addition to the usual statistics given under various headings, other facts have been briefly stated in the belief that the membership will be interested in having a more detailed knowledge of the general condition of the Society, and of the amount of work necessary at the present time to carry out its functions in a manner most useful to the membership.

MEMBERSHIP

The changes in membership are shown in the following table:

	Jan. 1st, 1912.			Jan. 1st, 1913.			Losses.				Additions.		Total.		
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignation.	Dropped.	Death.	Transfer.	Election.	Loss.	Gain.	Increase.
Honorary Members.	.....	8	8	.....	7	7	....	..	..	1	.....	.....	1	....	§1
Corresponding " .	.....	2	2	.....	2	2	....	..	..	..	.....	.....	....	....	....
Members.	623	2 305	2 928	633	2 432	3 065	....	9	5	51	*83	§119	65	202	137
Associate Members.	497	1 889	2 386	513	2 178	2 691	81	16	16	9	†102	‖325	122	427	305
Associates	80	98	178	76	103	179	2	5	2	3	‡2	11	12	13	1
Juniors	169	634	803	156	658	814	102	16	25	..	.....	‖154	143	154	11
Fellows	6	15	21	7	12	19	....	..	..	2	.....	.....	2	....	¶2
Totals	1 375	4 951	6 326	1 385	5 392	6 777	185	46	48	66	187	609	345	796	451

* 81 Associate Members, 1 Associate and 1 Junior.
† 1 Associate and 101 Juniors.
‡ 2 Juniors.
§ 3 Reinstatements.
‖ 1 Reinstatement.
¶ Decrease.

It will be noted that the net increase in membership for the year is 451.

With the last Annual Report of the Board a diagram showing the total membership each year on January 1st, from 1871 to 1912, was published. It may be found of interest, as showing the increase in total membership, to give the following figures for 10-year periods:

On January 1st, 1873, the total membership was 337:

For the 10 years	1873-1882 (inclusive) the total net increase was	382
" " " "	1883-1892 " " " " " "	890
" " " "	1893-1902 " " " " " "	1 103
" " " "	1903-1912 " " " " " "	4 065

The number of applications received during 1912 was 906: 670 for admission, and 236 for transfer.

The losses by death reported during the year number 66, and are as follows:

Honorary Members (1): George Wallace Melville.

Members (51): Charles Albert Allen, Thomas Chalkley James Baily Jr., Robert Ballard, Andrew Bell, Frederick Wagoner Bennett, Alfred Pancoast Boller, Daniel Seymour Brinsmade, Carl Waldemar Buchholz, Alfred Ellsworth Carter, James Edmund Childs, William Billings Clapp, Morris M. Defrees, Samuel Clarence Ellis, James Kennon Geddes, Stephen Samuel Haight, Julien Astin Hall, Charles Lewis Harrison, Benjamin Morgan Harrod, Arthur Powis Herbert, John Herron, James Breading Hogg, Horace E. Horton, William Bell White Howe, Thomas Moore Jackson, Edward Henry Keating, George Albert Kimball, Lewis Kingman, William Frederick Lockwood, Henry Fiddeman Lofland, James Riddle Maxwell, Frank Otis Melcher, David Spencer Merritt, Charles Augustine Miner, Edward Mohun, Charles Edward Moore, Charles James Morse, John Lawrence Power O'Hanly, LaFayette Olney, Walter A. Post, Joseph Allen Powers, William Haselton Puffer, Robert Leland Read, James Dix Schuyler, James Moore Shanley, Cecil Brunswick Smith, Joseph Shuter Smith, T. Guilford Smith, Julian Thornley, William Chattin Wetherill, George Howard White, Henry Fisher White.

Associate Members (9): Rowan Ayres, Donald Dean Colvin, Henry England Grimm, Walter Scott Hanna, John Hawkesworth, Antonio Esteban Mesa, William Madison Myers, Ulrich Taubenheim, James Hugh Wise.

Associates (3): Basil Henry Leather, James Dynan Newton, James Breckinridge Speed.

Fellows (2): Bernard N. Farren, Stephen Holman.

LIBRARY

The total contents of the Library and the increase during the year, are shown in the following statement:

	Total Contents.	Increase during 1912.
Bound volumes	21 907	1 282
Unbound volumes	42 132	2 415
Specifications	7 197	180
Maps, photographs and drawings	4 598	172
Total	75 834	4 049

Of these, 1 720 were donations received in answer to special requests; 94 were donations from publishers; 1 980 were donations received in regular course, and 255 were purchased.

The value of accessions to the Library during the year is as follows, each accession having been valued separately as received:

Donations and exchanges (estimated value)..	$2 423.99
255 volumes purchased (cost)...............	761.39
Binding 348 volumes.......................	412.00
Total	$3 597.38

The following amounts have been expended upon the Library during the year:

Purchases, subscriptions, and binding........	$1 173.39
Fixtures, supplies, express charges, etc........	239.06
Total	$1 412.45

The card index now contains about 83 000 cards.

During the year 67 new bibliographies (containing 2 537 separate references) have been made, copies of 14 searches made in previous years have been furnished, 6 of these having been brought up to date. The total cost of this work, $793.93, has been charged to those for whom it was undertaken.

The total attendance in the Reading Room and Library during the year was 3 967, but this does not include those who use the Library during the semi-monthly meetings.

For a statement in detail of the general work carried out in the Library, members are referred to the report of the Board for last year.

OFFICE WORK

The following summary may serve to place before the membership some idea of the volume of work required in the office.

During the year 1912 the following material was mailed from the Society House: 17 529 Letters; 27 800 Postals; 6 718 Forms; 20 442 Volumes of *Transactions;* 68 899 Numbers of *Proceedings;* 70 785 copies of Preliminary Lists; 64 604 Circulars; 7 077 copies of List of Members; 6 314 Volumes of Index to *Transactions;* 4 522 Separate Papers (about 1 000 packages); 895 copies of Memoirs (about 100 packages); 11 032 Bills; 6 600 Receipts; 319 Ballots; 642 Badges; and 321 Certificates of Membership; a total of 310 182 pieces, or an average of 1 024 pieces for every working day. Therefore, the average number of times the Society communicates with each of its members during the year is about 50.

During 1912 the total weight of material handled in the office and forwarded by mail, express, etc., was about 81½ tons, of which *Transactions, Proceedings,* and Indexes constituted about 72 tons.

The Bookkeeping Department now handles 7 000 separate accounts, and this number increases by 500 each year.

The checking, rewriting and briefing of the 900 applications received each year, including proof reading for the Preliminary Lists of Candidates, requires great care and much time.

The volume of the publications is shown under the next heading, but reference should be made here to the large amount of editorial work, and to the proof reading, preparation of illustrations, etc.

PUBLICATIONS

During the year, ten numbers of *Proceedings,* one volume of *Transactions* for 1911, and one for 1912, one volume of Index to *Transactions* and one List of Members, have been issued.

In *Proceedings* the list of references to current engineering literature has been continued, and has covered 148 pages and contained 6 409 classified references to 108 periodicals.

The stock of the various publications of the Society, kept on hand for the convenience of members and others, now amounts to 160 055 copies, the cost of which to the Society, for paper and press work only, has been $23 316.20.

During the year, 8 377 volumes of *Transactions* and 4 275 volumes of Index to *Transactions* have been bound for members and others in standard half-morocco and cloth bindings.

SUMMARY OF PUBLICATIONS FOR 1912.

	Issues.	Average Edition.	Total Pages.	Plates.	Cuts.
Transactions (Volumes LXXIV and LXXV)	2	6 875	1 772	75	283
Proceedings (monthly numbers)	10	7 100	2 586	140	194
Constitution and List of Members	1	7 400	277	...	1
Index to *Transactions*, Volumes 1 to LXXIV	1	7 200	412	...	...
Totals	14	.....	5 047	215	478

The cost of publications has been:

For Paper, Printing, etc., *Transactions* and *Proceedings*	$26 947.87
For Plates and Cuts	2 544.86
For Boxes, Mailing Lists, Copyright, and Sundry Expenses	940.97
For 9 125 Extra Copies of Papers and Memoirs	1 510.34
For List of Members	1 793.15
For Index to *Transactions,* Volumes I to LXXIV	1 960.44
Total	$35 697.63
Deduct amount received from sale of publications	4 924.28
Net expenditure for publications for 1912	$30 773.35

In the last Annual Report the Board announced briefly that a new system of publication would go into effect in 1912, the chief change being the issue of one volume only of *Transactions,* instead of four. In order to accomplish this "Bible" or "India" paper was to be used. Volume LXXV, just issued, is the result, and from the many expressions of commendation received, it appears that the change has been highly satisfactory to the membership, and that the prediction that it would be a material advance has been realized. In addition to the fine appearance of the letter-press of this Volume, and its compactness (more than 1 200 pages being less than one inch in thickness), it is economical. A comparison of the cost of this Volume, including the postage, with that of the same quantity of matter if published in the old way, but omitting consideration of the half-morocco and cloth bindings undertaken for members, shows an additional cost of only $432; whereas, if that binding is included in the comparison, the total cost of the Volume is $1 020 less than if published in the old way. Moreover, in future years, when the number of pages in the annual volume is greater than in this one, perhaps twice as great, the additional cost per page will be reduced, and the advantage of this method will be shown by a still greater reduction in the total cost.

The fact should also be pointed out that the saving to the individual members of the Society who have their *Transactions* bound, has, on this volume alone, aggregated $1 519, and that this saving will be fully twice as great in the future.

In addition to the issue of the *Proceedings* and *Transactions,* an Index to Volumes I to LXXIV (covering the entire issue of this publication to date) has been compiled during the year, and issued to all members.

In a general way it may be of interest to give some figures as to the amount of printed matter issued by the Society. The issue of Monthly *Proceedings* as well as *Transactions* was begun in January, 1896, and since that time every issue of the former has been mailed to the membership on the fourth Wednesday of the month. The total number of pages published since that date has been 60 167, an average of 3 539 pages per annum.

MEETINGS

During the year 29 meetings have been held, as follows: At the Annual Meeting, 2; at the Annual Convention, 5; and 22 other meetings held at the Society House.

At these meetings there were presented 25 formal papers, 6 of which were illustrated with lantern slides. There were also 10 papers published which were not presented for discussion at any meeting of the Society. The number of members and others who took part in the preparation or discussion of these papers was 281.

The Forty-fourth Annual Convention was held at Seattle, Wash.

The total attendance at the 29 meetings held was about 4 403. The registered attendance at the Annual Meeting was 838, and at the Annual Convention 155 (includes members only), but there were many members and guests present at all of these meetings who failed to register.

At each of the ordinary semi-monthly meetings held during the year Collations have been served and these have been paid for out of Society Funds, in accordance with the action of the Annual Meeting of 1912.

In addition to the above, the Society entertained two bodies of distinguished Foreign Engineers as follows:

By special request of the General Secretary of the Twelfth International Congress of Navigation, which Congress was held in Philadelphia, Pa., the visit of the delegates to the City of New York was made under the auspices of this Society. A Reception was given at the Society House on the evening of June 3d, 1912. Excursions were made on June 4th around the Harbor (as the guests of the City of New York), and on June 5th to the Terminal Improvement of the New York Central and Hudson River R. R., and to the subway work under construction in Fourth Avenue, Brooklyn. The delegates were entertained on this day at luncheon at the Manhattan Hotel.

On the evening of September 5th, 1912, a Reception was tendered to the Foreign Delegates of the Sixth Congress of the International Association for Testing Materials, which Congress was in session in New York City during the week beginning September 2d.

MEDALS AND PRIZES

For the year ending with the month of July, 1911, prizes were awarded as follows:

The Norman Medal to George Gibbs, M. Am. Soc. C. E., for his paper entitled "The New York Tunnel Extension of the Pennsylvania Railroad: Station Construction, Road, Track, Yard Equipment, Electric Traction, and Locomotives."

The Thomas Fitch Rowland Prize to B. H. M. Hewett and W. L. Brown, Members, Am. Soc. C. E., for their paper entitled "The New York Tunnel Extension of the Pennsylvania Railroad: The North River Tunnels."

The Collingwood Prize for Juniors to A. Kempkey, Jun. Am. Soc. C. E. (now Assoc. M. Am. Soc. C. E.), for his paper entitled "A Concrete Water Tower."

During the year two additional Prizes have been established by the Board. These prizes, which may be awarded annually for papers published in the *Transactions,* are to be known as the J. James R. Croes Medal, in honor of the first recipient of the Norman Medal, and the James Laurie Prize, in honor of the first President of the Society.

FINANCES

During the year $10 000 was paid on the principal of the Mortgage on the Society Property, reducing this debt to $115 000. An addition of $20 000 was also made to the Reserve Fund established two years ago, and the Society now has a Reserve Fund of $57 000, invested in non-taxable bonds of the City of New York, which yields somewhat more than 4%, the rate paid on the Mortgage debt. The Board has ordered that $10 000 be paid on the principal of the mortgage early in 1913, and that $20 000 be added to the Reserve Fund.

The following statement indicates clearly the prosperous financial condition of the Society.

On May 25th, 1895, when the purchase of property and the building of a new Society House was under consideration, the Board of Direction issued a statement giving the available assets as follows:

"The House, 127 East Twenty-third St. (estimate)	$60 000	
Securities in Safe Deposit (par value)	16 000	
Cash awaiting permanent investment	4 500	
Total	$80 500	
Mortgage on 127 East Twenty-third St.	16 000	
Amount available		$64 500"

On the same basis a similar statement as of December 31st, 1912, would read:

The Property, 220 West 57th St. (estimate)	$600 000	
Securities in Safe Deposit (par value)	57 000	
Cash awaiting permanent investment	44 000	
Total	$701 000	
Mortgage on 220 West 57th St.	115 000	
Total available assets		$586 000

Therefore the increase in available assets in 17½ years has been $521 500, or an average of about $30 000 per year.

The statement of May 25th, 1895, continues:

"The average income from all sources for the past five years has been $33 000."

We can now say:

The average annual income from all sources for the past five years has been $125 500.

The attention of members is invited to the Secretary's statement of receipts and disbursements, and to the general balance sheet which accompanies it.

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction.

CHAS. WARREN HUNT,
Secretary.

JANUARY 7TH, 1913.

GENERAL BALANCE SHEET, DECEMBER 31st, 1912.

Accompanying the Report of the Secretary.

Assets.

Three Lots (Actual cost, $189 632.11) (Estimated value)		$375 000.00
Society Building (cost)		170 955.59
Furniture (cost)		20 110.65
Publications on hand (inventoried cost)		23 316.20
New York City Non-Taxable Bonds (cost)		60 437.50
Library: Cash expended for books, etc.	$17 125.35	
Donations (estimated)	62 807.99	
		79 933.34
Due from Members		10 083.55
Due from Non-Members		662.29
Cash		44 074.04
		$784 573.16

Liabilities.

Dues for 1913 paid in advance	$28 107.55
Mortgage Debt and Loan	115 000.00
Fund invested in Society House, Lots, and Library*	27 190.78
Herbert Stewart Library Fund	1 997.50
Gen. Joseph G. Swift Library Fund	998.75
Surplus (including Reserve Fund of $57 441.25)	611 278.58
	$784 573.16

We have examined the books and accounts of the American Society of Civil Engineers, for the year ended December 31, 1912, and certify that the foregoing Balance Sheet is in accordance therewith, and, in our opinion, correctly states the condition of the Society's affairs, as shown by the books.

79 Wall Street, New York.
January 8th, 1913.

Marwick, Mitchell, Peat, & Co.,
Chartered Accountants.

* Compounding Dues Fund, $10 930.00; Norman Medal Fund, $1 000.00; Rowland Prize Fund, $1 222.50; Collingwood Prize Fund, $1 000.00; Fellowship Fund, $13 038.28

REPORT OF THE SECRETARY FOR THE

To the Board of Direction of the

Gentlemen:—I have the honor to present a statement of Receipts 31st, 1912. I also append a general balance sheet showing the condition

Receipts.

Balance on hand December 31st, 1911, in Bank, Trust Company, and in hands of Treasurer................		$46 363.95
Entrance Fees..............................	$15 290.00	
Current Dues...............................	74 367.66	
Past Dues..................................	2 997.04	
Advance Dues...............................	28 107.55	
Certificates of Membership.................	671.15	
Badges.....................................	3 120.85	
Sales of Publications......................	4 924.28	
Compounding Dues...........................	250.00	
Library....................................	661.84	
Annual Meeting.............................	500.10	
Binding....................................	2 945.94	
Interest...................................	2 737.20	
Miscellaneous..............................	150.51	
		136 724.12
		$183 088.07

YEAR ENDING DECEMBER 31ST, 1912.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

and Disbursements for the fiscal year of the Society, ending December
of the affairs of the Society.

Respectfully submitted,
CHAS. WARREN HUNT,
Secretary.

DISBURSEMENTS.

Salaries of Officers	$12 800.00	
Mileage of Directors	1 331.05	
Clerical Help	20 215.39	
Caretaking	1 910.64	
Publications	35 697.63	
Postage	9 146.39	
General Printing and Stationery	3 126.88	
Library	1 173.39	
Library Maintenance	239.06	
Badges	1 883.50	
Certificates of Membership	453.00	
Binding	7 364.18	
Prizes	166.10	
Convention	516.45	
Annual Meeting	1 254.95	
Maintenance of House	109.50	
Heat, Light, and Water	1 032.69	
Furniture	330.08	
Work of Committees	316.93	
Interest	4 647.86	
Current Business	2 889.46	
Petty Expenses	216.19	
Entertainment, Navigation Congress	1 194.13	
Entertainment, 6th Congress International Assoc. for Testing Materials	729.42	
Bond and Mortgage (Payment on Principal)	10 000.00	
Reserve Fund	20 269.16	
		$139 014.03
Balance on hand December 31st, 1912:		
In Union Trust Company	$9 909.82	
In Garfield National Bank	32 664.22	
In hands of Treasurer	1 500.00	
		44 074.04
		$183 088.07

REPORT OF THE TREASURER.

In compliance with the provisions of the Constitution, I have the honor to present the following report for the year ending December 31st, 1912:

Balance on hand December 31st, 1911			$46 363.95
Receipts from current sources, January 1st to December 31st, 1912			136 724.12
Payment of Audited Vouchers for Current Business, January 1st to December 31st, 1912		$108 744.87	
Payment on principal of bond and mortgage		10 000.00	
Purchase of bonds, Reserve Fund		20 269.16	
Balance on hand December 31st, 1912:			
In Union Trust Company	$9 909.82		
In Garfield National Bank	32 664.22		
In hands of the Treasurer	1 500.00		
		44 074.04	
		$183 088.07	$183 088.07

Respectfully submitted,

Jos. M. Knap,
Treasurer.

New York, January 7th, 1913.

ACCESSIONS TO THE LIBRARY

(From December 4th, 1912, to January 1st, 1913)

DONATIONS*

THE AMERICANS IN PANAMA.

By William R. Scott. Cloth, 8 x 5½ in., illus., 13 + 258 pp. New York, The Statler Publishing Company, 1912. $1.47.

This book, it is stated, is a non-technical review of events leading up to, and occurring during, the building of the Panama Canal, and particularly of the American engineers and laymen who have effected its construction in the broadest sense, by one who spent three months as an employee of the Isthmian Canal Commission. The scope of the book is limited to achievements on the Isthmus of Panama and only so much history of the Isthmus under the Spaniards and the French is given as will lend a perspective to the work of the Americans. The Contents are: The Land Divided—The World United; The Life Cost of the Canal; The Spanish in Panama; The Panama Railroad; The French in Panama; The Americans in Panama; The Roosevelt Impetus; Taking the Canal Zone; The Geography of Panama; Getting Under Way; The Canal Under Wallace; The Canal Under Stevens; The Canal Under Goethals; Locks and Dams; The Culebra Cut; Labor; Commissary—Quarters—Subsistence; Civil Administration; The Society of the Chagres; The Trade Outlook; Settling Our Account with Colombia; The Monroe Doctrine.

ELEMENTS OF HYDRAULICS:

A Text-Book for Secondary Technical Schools. By Mansfield Merriman, M. Am. Soc. C. E. Cloth, 7¾ x 5 in., illus., 6 + 156 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1912. $1.00.

The preface states that the author has attempted in this book to present the subject of hydraulics without the use of the higher mathematics, a knowledge of arithmetic, algebra, trigonometry, and elementary mechanics only being required on the part of the reader. The book, as stated, is intended particularly for the use of students in the upper classes of secondary technical schools, and while the essential principles and methods of hydraulics have been covered, certain topics have necessarily been inadequately treated or omitted altogether, the author's intention being to discuss only those topics which are of greatest importance in practical engineering. The Chapter headings are: Hydrostatics; Theoretic Hydraulics; Flow from Orifices and Tubes; Flow Through Pipes; Flow in Conduits and Rivers; Measurement of Water; Hydraulic Motors; Pumps and Pumping; Index.

ENGINEERING OF SHOPS AND FACTORIES.

By Henry Grattan Tyrrell. Cloth, 9½ x 6¼ in., illus., 17 + 399 pp. New York, McGraw-Hill Book Company, 1912. $4.00.

This book is a sequel of, and supplementary to, the author's "Mill Buildings." It is based, it is stated, on his personal observations, study, and experience in the arrangement, construction, and operation of shops and factories extending over a period of more than twenty years, and is intended as an aid to engineers, architects, draftsmen and students, as well as factory owners and employees. In the first chapter the author gives the standard rules of conduct and business which have been established and accepted by several of the leading engineering societies. This is followed by chapters on the economics of factory location and construction, and there are included several chapters on concrete buildings and their cost, together with descriptions of easy and effective methods for their surface treatment. The subjects, foundations, walls, roofing, etc., which are treated fully in the author's previous book, are only briefly mentioned here. Several of the chapters are said to have appeared previously in *The Engineering Magazine*, and the chapters on Heating and Air Washing were contributed by the Buffalo Forge Company and that on Artificial Lighting by the Westinghouse Electric Company. At the end of the book, the author has included an extensive bibliography of the subject. The Contents are: Engineers and Their Services; Manufacturing District; Economics of Factory Construction; Example of Preliminary Design; Gen-

* Unless otherwise specified, books in this list have been donated by the publishers.

eral Design; Selection of Building Type; Wood and Steel Framing; Concrete Buildings; Concrete Surface Finish; Cost of Reinforced Concrete Buildings; Comparative Cost of Wood, Reinforced Concrete, and Steel Buildings; Foundations; Ground Floors; Upper Floors; Concrete Upper Floors; Walls, Partitions and Openings; Roofs and Roofing; Special Buildings—Notes on; Storage Pockets and Hoisting Towers; Factory Heating; Air Washing Systems; Drainage of Industrial Works; Water Supply and Storage Tanks; Steel Chimneys; Fire Protection; Cranes; Yards and Transportation; Estimates; Construction; Welfare Features; Standard Buildings, Bibliography; Index.

MODERN ORGANIZATION:

An Exposition of the Unit System. By Charles DeLano Hine. (Works Management Library.) Cloth, 7½ x 5 in., 110 pp. New York, The Engineering Magazine Co., 1912. $2.00.

The subject-matter contained in this volume appeared originally as a series of articles in *The Engineering Magazine* from January to July, 1912. It is here presented, it is stated, as a comprehensive definition of the author's unit system of management, a complete unfolding of his doctrine and practice (which, it is said, is largely mental suggestion), as applied by him on the Harriman Lines, the most extensive railway system in the world. The Contents are: The Unit System on the Harriman Lines; Operation of the Unit System; Broadening the Ideals of Line Supervision; Over-Specialization; Fallacies of Accounting; Supplies and Purchases; Line and Staff; Genesis and Revelation of Organization.

FOURTH NATIONAL CONFERENCE ON CITY PLANNING:

Proceedings, Boston, Massachusetts, May 27-29, 1912. Cloth, 9½ x 6¼ in., 10 + 232 pp. Boston, 1912. $2.00. (Donated by the National Conference on City Planning.)

This Conference, like those of 1909, 1910, and 1911, was held, it is stated, for the personal exchange of information and ideas on the subject of city planning by architects, engineers, business men, and others interested in such work, in order that the principles and details of such planning may become better known and the work better done. This volume contains the chief papers read at the Conference, together with summaries of the discussions on those papers. A partial list of the Contents is: The Progress in City Planning, by Frederick L. Olmsted; The Meaning of City Planning, by Arnold W. Brunner; The Attitude of the Engineer Toward City Planning, by George F. Swain; Paying the Bills for City Planning, by Nelson P. Lewis; Paying the Bills for City Planning from a Boston Viewpoint, by James A. Gallivan; Round Table Talks; The Problem of the Blighted District, by J. Randolph Coolidge, Jr.; The Public Street Systems of the Cities and Towns About Boston in Relation to Private Street Schemes, by Arthur A. Schurtleff; Street Planning in Newton, by Edwin H. Rogers; Street Planning in Watertown, by Wilbur F. Learned; The Legislation Necessary for Intelligent City Planning; The Regulation of the Height of Fireproof Commercial Buildings, by Arthur C. Comey; How a Worcester, Mass., Bank Discourages the "Three-Decker" House, by Alfred L. Aiken; Practical *versus* Ideal City Planning, by Amos L. Schaeffer; Popularizing the City Planning Principle; The Control of Municipal Development by the "Zone System" and Its Application in the United States, by B. Antrim Haldeman; etc., etc.

THE CURTISS AVIATION BOOK.

By Glenn H. Curtiss and Augustus Post. Cloth, 7½ x 5 in., illus., 10 + 307 pp. New York, Frederick A. Stokes Company, 1912. $1.35.

The subject-matter of this book is divided into six parts. Part I, Boyhood and Early Experiments, by Augustus Post, is devoted to the story of Mr. Curtiss' early life and his experiments with motors, motor-cycles, and balloons. In Parts II and III, Mr. Curtiss describes his first and his chief flights, together with his work and experiments on aeroplanes and hydroaeroplanes. In Part IV, The Real Future of the Aeroplane, Mr. Curtiss discusses its future uses and the problems of aviation; Capt. Paul W. Beck, U. S. A., has a chapter on the aeroplane as applied to the army, the naval side being discussed in another chapter by Lieut. Theodore G. Ellyson, U. S. N., and aviation as a future sport is described by Mr. Post. Part V, Every-Day Flying for Professional and Amateur, with chapters by Messrs. Curtiss, Post, and Hugh Robinson, is devoted to the teaching of aviation and to the operating of the aeroplane and hydroplane. In Part VI, Mr. Post discusses the Curtiss pupils and gives a description of the Curtiss aeroplane and motor.

THE METALLOGRAPHY OF IRON AND STEEL.

By Albert Sauveur. Cloth, $10\frac{3}{4}$ x $7\frac{1}{2}$ in., illus., various paging. Cambridge, Mass., Sauveur and Boylston, 1912.

The preface states that, in this book, the author has endeavored to present, in a scientific and practical manner, a well-balanced, specific and comprehensive treatise on the subject of the metallography of iron and steel, which is intended for the student, the teacher, the manufacturer and user of iron and steel, the general reader, and the expert. He describes at length, it is said, the structure of metals, the processes, manipulations, and apparatus used successfully in his own laboratory and in those of other workers. At the end of each chapter examination questions on the subject discussed in that chapter are given, and the book is fully illustrated. The Contents are: Introduction; The Industrial Importance of Metallography; Apparatus for the Metallographic Laboratory; Pure Metals; Pure Iron; Wrought Iron; Low Carbon Steel; Medium High and High Carbon Steel; Impurities in Steel; The Thermal Critical Points of Iron and Steel; Their Occurrence; Their Causes; Their Effects; Cast Steel; The Annealing of Steel; The Hardening of Steel; The Tempering of Hardened Steel; Theories of the Hardening of Steel; The Cementation and Case Hardening of Steel; Special Steels; General Considerations; Constitution, Properties, Treatment, and Uses of Most Important Types; Cast Iron; Impurities in Cast Iron; Malleable Cast Iron; Constitution of Metallic Alloys; Equilibrium Diagram of Iron-Carbon Alloys; The Phase Rule; Appendix I, Manipulations and Apparatus; Appendix II, Nomenclature of the Microscopic Constituents; Index.

SUCCESSFUL HOUSES AND HOW TO BUILD THEM.

By Charles E. White, Jr. Cloth, $7\frac{1}{4}$ x $5\frac{1}{4}$ in., illus., 6 + 520 pp. New York, The Macmillan Company, 1912. $2.00.

In this book, and without giving cost data, the author describes, for the benefit of the prospective owner and builder, the best location and arrangement for various types of houses, together with building specifications and other legal documents in connection therewith, the construction of a house in detail, and the various useful apparatus and appliances necessary to a successful house. The last chapter is devoted to modern garage design and garage apparatus. The Contents are: Why Build a Home; Choosing the Site; How to Know the Architectural Styles; The Little Details That Attract; Owner, Architect, and Contractor; Planning the Rooms; Specifications Explained; A Chapter on Legal Documents; Excavation and Foundation; Advantages of a Frame House; Exterior Finish; Houses of Masonry; How to Build a Fireproof House; Carpentry and Cabinet Work; The Importance of a Good Roof; Plumbing That Is Sanitary; Water and Sewer Pipes; Latest Types of Plumbing Fixtures; Little Details of Good Plumbing; Sewage Disposal in the Country; Efficient Heating Methods; Plastering; Inside and Outside; Finishing Touches; Painting and Glazing; Useful Apparatus and Appliances; Gas and Electric Lighting; Practical Hardware for the House; Handy House Devices; Remodeling; Making an Old House New; Sensible Types of American Houses; Garages and Garage Apparatus; Index.

FIRE PREVENTION.

By Edward F. Croker. Cloth, 8 x $5\frac{1}{4}$ in., illus., 10 + 354 pp. New York, Dodd, Mead & Company, 1912. $1.50.

The fighting of many fires and the study of the subjects of fire prevention and fire extinguishment during the author's years of connection with the Fire Department of New York City, has led, it is stated, to the recommendations and plans presented by him in this book, in which he discusses the keeping down of fire waste from many angles, and describes, it is stated, those things which are vitally necessary to the reduction of fire loss, such as, fireproof construction, automatic fire alarms and devices for extinguishing fires, fire drills, fire departments, etc. The Chapter headings are: Prevention of Fire in the Dwelling and Small Town; Further Measures for Protection in the House—The Department of a Small Town; The Protection of Factories, Loft Buildings, and other Large Structures; Protection of Life in Large Buildings; Further Measures for Protecting Life; Sprinklers and Sprinkler Systems; Other Alarm Devices in Buildings; The Fire Department of a Large City; The Development of the New York Fire Department and Some of its Defects; The Nature and Value of High Pressure; The Modern Fireboat: Its Nature and Use; High Buildings and Steel Construction; Fire Prevention Bureaus and Fire Marshals; Incendiarism and Arson; Law Making and Fire Preventive Work Along Legal and Other Lines; Index.

MINERALOGY:

An Introduction to the Theoretical and Practical Study of Minerals. By Alexander Hamilton Phillips. Cloth, 9 x 6 in., illus., 8 + 699 pp. New York, The Macmillan Company, 1912. $3.75.

The preface states that the object of this book is to bring together, in concise form, for the use of the beginner, the facts and basic principles of the several branches of Mineralogy. The subject-matter has been divided into three parts: Part I, Crystallography, in which the 32 types have been described. Only graphical methods of solving the problems, after the measurement of a crystal, are given, it is stated, the mathematical solutions being left to the more advanced courses. In Part II, Descriptive Mineralogy, some 225 mineral species are described including their crystallization, optical properties, decomposition, products, genesis, occurrence, uses, and synthesis. Part III, Determinative Mineralogy, contains descriptions of the instruments and reagents, and gives determinative tables and chemical tests, used for the identification of the mineral species, including, it is stated, all minerals with the exception of some very rare species found only in one locality, and in many cases even these are given.

Gifts have also been received from the following:

Adams, Mass.-Chf. Engr., Fire Dist. 33 pam.
Annan, C. L. 4 pam.
Arizona, Univ. of-Agri. Exper. Station. 2 pam.
Arnold, Bion J. 1 pam.
Associated Metal Lath Mfrs. 1 pam.
Boston, Mass.-City Auditor. 2 bound vol.
Boston, Mass.-City Council. 1 bound vol.
Buffalo, N. Y.-City Clerk. 1 pam.
California-R. R. Comm. 1 pam.
California-State Conservation Comm. 5 pam.
California-State Forester. 1 pam.
Canada-Dept. of Marine and Fisheries. 1 vol.
Canada-Dept. of Mines. 1 vol.
Chicago, Ill.-Bureau of Statistics. 1 bound vol.
Chicago, Ill.-Harbor and Subway Comm. 1 pam.
Chicopee, Mass.-City Clerk. 7 bound vol.
Cincinnati, Ohio-Supt. of Water-Works. 1 vol.
Colorado, Univ. of. 1 pam.
Columbia Univ. 1 vol.
Concord, N. H.-City Clerk. 19 vol., 13 pam.
Concord, N. H.-City Engr. 6 pam.
Cornell Univ. 1 pam.
Dist. of Columbia-Engr. Commr. 1 vol.
East Indian Ry. Co. 1 pam.
Eiffel, G. 1 pam.
Farnham, A. B. 9 pam.
Flynn, George A. 3 bound vol.
Germany-Kaiserliche Generaldirektion der Eisenbahnen in Elsass-Lothringen. 1 vol.
Great Indian Peninsula Ry. Co. 1 pam.
Haverhill, Mass.-City Engr. 1 pam.
Illinois-State Rivers and Lakes Comm. 8 pam.
Institution of Mech. Engrs. 1 vol.
Iowa-Auditor of State. 1 vol.
Iowa-Board of R. R. Commrs. 1 bound vol.
Kansas City, Mo.-Board of Public Welfare. 1 pam.
Kansas City, Mo.-Comptroller. 1 pam.
Lake Superior Min. Inst. 1 vol.
London, England-Town Clerk, Public Health Dept. 2 pam.
Margate, England-Town Clerk. 2 bound vol.
Massachusetts-Bureau of Statistics. 1 bound vol.
Massachusetts-Mount Everett State Reservation Comm. 1 pam.
Massachusetts-State Board of Health. 1 bound vol.
Merrill, J. Evarts. 7 pam.
Municipal Engrs. of the City of New York. 1 pam.
Min. Soc. of Nova Scotia. 1 pam.
Minneapolis, Minn.-City Clerk. 17 bound vol.
Missouri, Univ. of-School of Mines and Metallurgy. 1 pam.
Montreal, Que.-Medical Officer of Health. 1 pam.
National Civic Federation. 4 pam.
New Mexico-State Corporation Comm. 1 pam.
New York City-Board of Water Supply. 2 pam.
New York State-Clerk of Senate. 1 bound vol.
North Carolina-State Board of Health. 6 pam.
Ohio State Univ. 1 vol., 1 pam.
Oregon-R. R. Comm. 1 pam.
Pasadena, Cal.-City Auditor. 9 pam.
Peoria, Ill.-City Clerk. 13 pam.
Philadelphia, Pa.-Dept. of Public Works. 4 pam.
Philippine Islands-Bureau of Public Works. 1 pam.
Philippine Islands-Weather Bureau. 2 pam.
Portland, Me.-Commr. of Public Works. 9 pam.
Poughkeepsie, N. Y.-City Chamberlain. 9 bound vol.
Rensselaer Polytechnic Inst. 2 pam.
Richmond, Fredericksburg & Potomac R. R. Co. 1 pam.
Roche, Wm. J. and Wm. C. 1 pam.
Royal Inst. of British Archts. 1 vol.
Royal San. Inst. 1 bound vol.
Saginaw, Mich.-City Engr. 14 pam.
St. Louis, Mo.-Municipal Reference Librarian. 3 bound vol., 17 vol.
Seattle, Wash.-City Comptroller. 1 pam.
Sheffield, England-City Engr. and Surv. 3 pam.

Smithsonian Institution. 3 pam.
Soc. for the Promotion of Eng. Education. 1 bound vol.
Soc. of Constructors of Federal Bldgs. 1 pam.
Sooysmith, Charles. 23 bound vol., 10 vol., 25 pam.
South Australia-Commr. of Rys. 1 pam.
South Dakota Soc. of Engrs. and Survs. 1 pam.
Stevens, L. E. 1 pam.
Stone & Webster. 1 pam.
Tasmania-Commr. of Govt. Rys. 1 pam.
Texas, Univ. of. 1 vol.
Tibbetts, Fred H. 1 vol.
Tyrrell, H. G. 1 pam.
Union Coll. 1 vol.
U. S.-Bureau of Chemistry. 1 pam.
U. S.-Bureau of Yards and Docks. 1 pam.
U. S.-Chf. of Engrs. 13 specif.
U. S.-Forest Service. 2 pam.
U. S.-Geol. Survey. 1 vol.
U. S.-Isthmian Canal Comm. 1 bound vol., 2 vol., 1 pam.
U. S.-Office of Public Roads. 1 pam.
Western Maryland Ry. Co. 1 pam.
Whitney, F. O. 26 pam.
Wilkerson, T. J. 2 vol.
Wisconsin-Geol. and Natural History Survey. 1 bound vol.
Wyoming-State Geologist. 1 pam.

BY PURCHASE

Analysis of Paint and Varnish Products. By Clifford Dyer Holley. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1912.

Building Stones and Clay-Products: A Handbook for Architects. By Heinrich Ries. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1912.

Light, Photometry and Illumination: A Thoroughly Revised Edition of Electrical Illuminating Engineering. By William Edward Barrows. McGraw-Hill Book Co., New York and London, 1912.

A Text-Book of Rand Metallurgical Practice Designed as a "Working Tool" and Practical Guide for Metallurgists Upon the Witwatersrand and Other Similar Fields. By C. O. Schmitt. Vol. 2. J. B. Lippincott Co., Philadelphia; Charles Griffin & Company, Ltd., London.

Standard Methods for the Examination of Water and Sewage. American Public Health Association, Laboratory Section. Second Edition. American Public Health Association, New York, 1912.

Public Health: Papers and Reports Presented at the Thirty-sixth and Thirty-seventh Annual Meetings of the American Public Health Association, Vol. 34, August 25th–28th, 1908; Vol. 35, October 19th–22d, 1909. American Journal of Public Hygiene, Columbus, Ohio, 1909–10.

SUMMARY OF ACCESSIONS

(From December 4th, 1912, to January 1st, 1913)

Donations (including 70 duplicates)	416
By purchase	7
Total	423

MEMBERSHIP

ADDITIONS

(From December 6th, 1912, to January 2d, 1913)

MEMBERS

		Date of Membership.
Boardman, Howard Edward. Oficina del Subterraneo, Ferro Carril de Oeste, Estacion Once, Buenos Aires, Argentine Republic	Jun. Assoc. M. M.	Oct. 7, 1902 Jan. 4, 1905 Oct. 1, 1912
Bowman, Daniel Wheeler. Chf. Engr., The Phœnix Iron Co., Phœnixville, Pa.		Dec. 3, 1912
Brown, William Nelson. Care, Brown & Clarkson, Southern Bldg., Washington, D. C.		Sept. 3, 1912
Fuller, Weston Earle. Cons. Civ. Engr. (Hazen & Whipple), 103 Park Ave., New York City	Assoc. M. M.	June 7, 1905 Dec. 3, 1912
Ide, William Stone. Res. Engr., Lockwood, Greene & Co., Care, The Studebaker Corporation, Detroit, Mich.		Dec. 3, 1912
Kimball, Joseph Harris. Designing Engr., Commrs. of Sewerage, 605 Equitable Bldg., Louisville, Ky.		Dec. 3, 1912
Perez Castro, Lorenzo. Jefe de la Zona VI. de Vias y Edificios, 1ª del General Prim 1, City of Mexico, Mexico		Dec. 3, 1912
Raff, Henry Gottlieb. Chf. Designing Engr., Concrete Steel Eng. Co., Park Row Bldg., New York City		Dec. 3, 1912
Reedy, Oliver Thomas. Supt., Royal Basin Min. & Milling Co., Maxville, Mont.	Assoc. M. M.	June 6, 1906 Dec. 3, 1912
Southworth, Edward Augustus. County Engr. of Hawaii, Hilo, Hawaii	Assoc. M. M.	July 1, 1909 Dec. 3, 1912
Williams, Francis Charles. City Engr., Sheridan, Wyo.		Dec. 3, 1912
Wolfe, Frank Gordon. Chf. Engr., Scranton Coal Co., Elk Hill Coal & Iron Co., and Price-Pancoast Coal Co.; Cons. Engr., N. Y., O. & W. R. R., 513 Board of Trade Bldg., Scranton, Pa.		Dec. 3, 1912

ASSOCIATE MEMBERS

Brainard, Normand Daggett. Supt. in Chg., Constr., The A. Bentley & Sons Co., 355 Stanton Ave., Springfield, Ohio		Dec. 3, 1912
Brown, David Harell. Alpine, N. J.	Jun. Assoc. M.	Feb. 4, 1908 Oct. 29, 1912
Caldwell, John Worde. Rd. Overseer, Dist. of Honolulu, Honolulu, Hawaii	Jun. Assoc. M.	Aug. 31, 1909 Dec. 3, 1912
Canaga, Gordon Byron. Scio, Ohio		Oct. 1, 1912
Curley, James Francis. Contr. Engr., Pittsburgh Steel Products Co., 959 Frick Annex, Pittsburgh, Pa.		Dec. 3, 1912

ASSOCIATE MEMBERS (*Continued*)		Date of Membership.
DUNHAM, ROBERT MOORE. Supt., Gen. Const. Co., 1116 Jarvis St., Fort Worth, Tex.		Dec. 3, 1912
DURBIN, WILLIAM HOWARD. Supt. of Filtration, Evansville, Ind.		Dec. 3, 1912
FOOTE, FRANCIS SEELEY, JR. Associate Prof. of R. R. Eng., Univ. of California, Berkeley, Cal.		Dec. 3, 1912
FORTER, SAMUEL ALEXANDER. Const. Engr., James A. Green & Co., Inc., American Falls, Idaho.		Oct. 1, 1912
FOSS, JAMES CALVIN, JR. Chf. Engr., Kahului R. R., Kahului, Maui, Hawaii.	Jun. Assoc. M.	April 6, 1909 Oct. 1, 1912
GORHAM, FRED ALLEN. Asst. Engr., U. S. Reclamation Service, Pompeys Pillar, Mont.		Dec. 3, 1912
HALL, HARRY RUTLEDGE. Asst. Engr., Bureau of San. Eng., State Dept. of Health, 6 East Franklin St., Baltimore, Md.		Dec. 3, 1912
HOFF, CARL PORTER. Asst. City Engr., St. Joseph, Mo.		July 9, 1912
LANGLOIS, AMEDEE. 455 Besserer St., Ottawa, Ont., Canada.		Sept. 3, 1912
OSBORN, IRWIN SELDEN. Engr. in Chg., Municipal Reduction Plant, Dept. of Public Service, 18 Seventeenth Ave., Columbus, Ohio.		Dec. 3, 1912
PAGON, WILLIAM WATTERS. With J. E. Greiner, Cons. Engr., Fidelity Bldg. (Res., 1301 St. Paul St.), Baltimore, Md.	Jun. Assoc. M.	Sept. 3, 1907 Dec. 3, 1912
PATTERSON, EARL. Engr., U. S. Reclamation Service, Selden, N. Mex.	Jun. Assoc. M.	April 30, 1907 Dec. 3, 1912
SAYFORD, NED HENSEL. With George W. Fuller, 170 Broadway, New York City (Res., Monroe, N. Y.)		Dec. 3, 1912
SCHABERG, BENJAMIN FRANKLIN. Engr. of Constr., Sewer Dept., 3830 Ashland Ave., St. Louis, Mo.		Dec. 3, 1912
SILVERTON, FRANCIS. Supt., Macdonell Gzowski & Co., P. O. Box 1723, Vancouver, B. C., Canada.		July 9, 1912
STEIN, MILTON FREDERICK. Asst. Engr., Chester & Fleming, 1111 Union Bank Bldg., Pittsburgh, Pa.		Dec. 3, 1912
STROUT, GALE STANLEY. 159 Lake St., Oakland, Cal.	Jun. Assoc. M.	Mar. 31, 1908 Sept. 3, 1912
THORLEY, IRA OTIS. With H. S. Crocker, Cons. Engr., 308 Tramway Bldg., Denver, Colo.		Oct. 29, 1912
TOLL, ASAHEL CLARK. Bayamon, Porto Rico.	Jun. Assoc. M.	Mar. 5, 1907 Oct. 29, 1912
VILLA, MIGUEL. Engr. in Chg. of Havana Harbor Work for the Compania de los Puerto de Cuba, O'Reilly 41, Havana, Cuba.	Jun. Assoc. M.	Oct. 6, 1908 Dec. 3, 1912
VOORHES, KIMBROUGH ENOCH. Arizona Copper Co., Morenci, Ariz.		Dec. 3, 1912

ASSOCIATE MEMBERS (*Continued*) — Date of Membership.

WESTON, THOMAS ISAAC. Pres. and Treas., Weston & Brooker Quarry Co. and Weston & Brooker, Contrs., 704 National Loan and Exchange Bank Bldg., Columbia, S. C. .... Dec. 3, 1912

ASSOCIATES

PARRY, FREDERICK HUGH. Eng. Contr., 400 Wyoming Ave., Kingston, Pa. .... Oct. 29, 1912

JUNIORS

BELDEN, GEORGE ALLYNE. Draftsman, Office of Chf. Engr., Cent. of Ga. Ry., Y. M. C. A. Bldg., Savannah, Ga. .... Oct. 29, 1912
BUCK, WALTER VAN. City Engr., Junction City, Kans. .... July 9, 1912
FISHER, GEORGE JOSEPH. Draftsman, South San Joaquin Irrig. Dist., Coyote, Cal. .... Dec. 3, 1912
JAMES, JOHN RAYMOND. Junior Engr., U. S. Lake Survey, Old Custom House, Detroit, Mich. .... Dec. 3, 1912
KHACHADOORIAN, HAROOTUN HOVHANNES. 34 St. Anne St., Quebec, Que., Canada .... Dec. 3, 1912
PERRIN, LESTER WILLIAM. Draftsman, G. T. Ry., 114 Dowling Ave., Toronto, Ont., Canada .... Dec. 3, 1912
SLEPPY, KIRBY BALDWIN. Instrumentman, Los Angeles Aqueduct Power Dept., 1134 Central Bldg., Los Angeles, Cal. .... Dec. 3, 1912
THACKWELL, HENRY LAWRENCE. Chf. Engr., Lake Chelan Land Co., 722 Leary Bldg., Chelan, Wash. .... Dec. 3, 1912
THOMAS, FRANKLIN. Draftsman, Alabama Power Co., Birmingham, Ala. .... Dec. 3, 1912
WADE, NEWTON BENJAMIN. City Engr., Millville, N. J. .... Dec. 3, 1912
YEREANCE, ALEXANDER WOODWARD. Engr. with MacArthur Bros. Co., Clanton, Ala. .... Dec. 3, 1912

RESIGNATIONS

MEMBERS — Date of Resignation.

FOLWELL, AMORY PRESCOTT .... Dec. 31, 1912
GARDNER, EDMUND LEBRETON .... Dec. 3, 1912
HODGES, GILBERT .... Dec. 3, 1912
MCCLINTOCK, WILLIAM EDWARD .... Dec. 31, 1912
ORANGE, JAMES .... Dec. 3, 1912
PAYNE, WILLIAM ARTHUR .... Dec. 31, 1912
RAASLOFF, HARALD DE .... Dec. 31, 1912

ASSOCIATE MEMBERS

CHAMBERS, HENRY WICK .... Dec. 31, 1912
DERBY, CHESTER CAWTHORNE .... Dec. 31, 1912
EGAN, LOUIS HENRY .... Dec. 31, 1912

ASSOCIATE MEMBERS (*Continued*)

	Date of Resignation.
Howalt, Wilhelm Jens Christian	Dec. 31, 1912
Jackson, Granbery	Dec. 31, 1912
Jouett, Henry Detrick	Dec. 31, 1912
Locke, William Willard	Dec. 31, 1912
Robbins, Hallet Rice	Dec. 31, 1912
Robinson, Frank Miner	Dec. 31, 1912
Shire, Moses Edmund	Dec. 31, 1912
Stenger, Ernest	Dec. 31, 1912
Stout, Homer Harding	Dec. 31, 1912
Toltz, Max	Dec. 31, 1912
Vogt, John Henry Leon	Dec. 3, 1912
Wood, George Roy	Dec. 31, 1912

ASSOCIATES

Bouton, Harold	Dec. 3, 1912
Green, Howard Burkhardt	Dec. 31, 1912
Ray, David Heydorn	Dec. 31, 1912

JUNIORS

Chandler, Horace Edward	Dec. 31, 1912
Croasdale, Laurence Brodhead	Dec. 31, 1912
Cunningham, William Augustine	Dec. 31, 1912
Lowry, John, Jr.	Dec. 31, 1912
Maglott, George Frederick	Dec. 31, 1912
Nitchie, Francis Raymond	Dec. 31, 1912
Payne, George Amos	Dec. 31, 1912
Stewart, Walter Phelps	Dec. 31, 1912

DEATHS

Allen, Charles Albert. Elected Member, June 4th, 1879; died December 9th, 1912.

Baily, Thomas Chalkley James, Jr. Elected Member, October 4th, 1905; died December 7th, 1912.

Ballard, Robert. Elected Member, September 1st, 1880; died November 22d, 1912.

Bell, Andrew. Elected Member, September 5th, 1883; died October 23d, 1912.

Boller, Alfred Pancoast. (*Vice-President.*) Elected Member, December 4th, 1867; died December 9th, 1912.

Defrees, Morris M. Elected Member, March 3d, 1880; died October 16th, 1912.

Grimm, Henry England. Elected Junior, October 2d, 1900; Associate Member, November 4th, 1908; died December 12th, 1912.

HAWKESWORTH, JOHN. Elected Junior, September 6th, 1904; Associate Member, November 4th, 1908; died December 10th, 1912.

SHANLY, JAMES MOORE. Elected Member, July 6th, 1887; died November 28th, 1912.

SPEED, JAMES BRECKINRIDGE. Elected Associate, May 2d, 1888; died July 7th, 1912.

THORNLEY, JULIAN. Elected Associate Member, January 2d, 1901; Member, September 5th, 1905; died December 21st, 1912.

Total Membership of the Society, January 2d, 1913,
6 778.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(December 5th, 1912, to January 1st, 1913)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Elektrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1. 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $4.

(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England, 6d.
(69) *Der Eisenbau*, Leipzig, Germany.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Mining and Engineering World*, Chicago, Ill., 10c.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga., 10c.

LIST OF ARTICLES

Bridges.

The Ohio River Highway Bridge at Sewickley.* V. R. Covell. (58) Oct.
Bridging of the Gamtoos River.* (From *South African Railway Magazine*.) (87) Dec.
A Highway Bridge with Provision for a Future Drawspan; Trail, B. C.* E. E. Howard. (13) Dec. 5.
The New Pontoon Bridge at Constantinople.* (12) Dec. 6.
Substructure of the East Haddam Bridge, Special Reinforced Concrete Girder Abutment on Eccentric Columns and a Reinforced Concrete Pivot Pier with Three Tiers of Pockets.* (14) Dec. 7.
City Bridge Floors under Heavy Traffic. (14) Dec. 7.
The New O.-W. R. & N. Bridge at Portland, Oregon.* W. P. Hardesty. (13) Dec. 12.
The Glen Loch Bridge Wreck.* (13) Dec. 19.
Larimer Ave. and Atherton Ave. Concrete Arch Bridges, Pittsburgh.* (13) Dec. 19.

*Illustrated.

Bridges—(Continued).

Supporting a Large Derrick from a Steel Bridge Tower.* (14) Dec. 21.
Large Reinforced Concrete Bridges in Pittsburgh.* N. S. Sprague. (Abstract of paper read before the National Assoc. of Cement Users.) (14) Dec. 21.
Constructional and Economic Features of Recent Concrete Bridge Work in Pittsburgh, Pa.* (86) Dec. 25.
A Successful Floating Concreting Plant for Bridge Construction.* Clement E. Chase. (86) Dec. 25.
The Electrical Equipment of the Hawthorne Avenue Bridge, Portland, Oregon.* (96) Dec. 26.
The 523½-Ft. Steel Arch Bridge over the Sanaga River, Cameroon, South Africa.* K. A. Muellenhoff. (13) Dec. 26.
Engineering Problems in the Foundations for a Cleveland Viaduct. A. M. Felgate. (Paper read before the Cleveland Eng. Soc.) (62) Dec. 30.
Warthebrücke bei Neustadt i. Posen.* Meyer. (40) Nov. 30.
Die neue Weichselbrücke bei Marienwerder.* Rhotert. (102) Dec. 1.

Electrical.

Yellow Flame Arcs.* Maurice Solomon. (77) Nov.
Power Generation and Distribution in the Clyde Valley Electrical Power Company's Area.* David A. Starr. (77) Nov.
Homopolar Generators.* Ernest W. Moss and J. Mould. (Abstract.) (77) Nov.
The Choice of Material for Overhead Line Conductors.* E. V. Pannell. (77) Nov.
High-Voltage Testing Transformers.* R. G. Parrott. (Abstract.) (77) Nov.
An Account of Some Experiments Made with Various Wireless Telegraphy Transmitters and a Complete Description of an Inexpensive Apparatus for Use Over Short Distances. Philip R. Coursey. (Abstract.) (77) Nov.
The Transmission of Electrical Energy by Direct-Current on the Series System.* J. S. Highfield. (77) Nov.
Condensers in Series with Metal Filament Lamps.* A. W. Ashton. (77) Nov.
The Four-Terminal Conductor and the Thomson Bridge. Frank Wenner. (Abstract from *Bulletin*, Bureau of Standards.) (73) Nov. 29.
Armature Reaction in Lap-Wound Machines. W. Lulofs. (73) Serial beginning Nov. 29.
The Loading of Submarine Telephone Cables. J. G. Hill. (26) Serial beginning Nov. 29.
Comparative Tests on High-Tension Suspension Insulators.* P. W. Sothman. (42) Dec.
High Frequency Tests of Line Insulators.* L. E. Imlay and Percy H. Thomas. (42) Dec.
A Method of Improving the Sensitiveness of the Telephone Receiver as a Detector in Alternating Current Null Measurements. Phillips Thomas. (3) Dec.
The Economical Speed Control of Alternating-Current Motors Driving Rolling Mills.* F. W. Meyer and Wilfred Sykes. (42) Dec.
Application of the Selenium Cell to Photometry.* A. H. Pfund. (Abstract from the *Physical Review*.) (73) Dec. 6.
Erecting Tall Wireless Towers at Arlington.* (14) Dec. 7.
Reinforced Concrete Telegraph Pole Tests. (14) Dec. 7.
Design of Piping for Transformer Oil, Air and Cooling Water.* Fred Buch. (27) Dec. 7.
The Reduction in the Cost of Electric Lighting Abroad.* (13) Dec. 12.
The Oil-Drop Method of Studying Electrical Phenomena in Gases. R. A. Millikan. (Abstract of paper read before the Am. Electrochemical Soc.) (73) Dec. 13.
Some Considerations in Connection with Electrically Driven Non-Reversing Mills. S. H. Eckmann. (73) Dec. 13.
Electric Cranes in Iron and Steel Works.* H. H. Broughton. (73) Dec. 13.
Electric Control Gear for Iron and Steel Works.* J. M. L. Slater and F. C. Hall. (73) Dec. 13.
The Selection and Care of Electrical Machinery in Steelworks.* J. Arthur Sykes. (73) Dec. 13.
Electric Furnaces in the Manufacture of Steel. Edward F. Law. (73) Dec. 13.
Induction Furnaces and Their Relation to the Steel Industry.* Joh. Härden. (73) Dec. 13.
The Electric Arc Furnace in Steel Production.* W. S Gifford. (73) Dec. 13.
Electrical Plant for Driving the Reversing Rolling-Mill at the Skinningrove Ironworks.* (73) Dec. 13.
The Shaw Electric Monorail System of Travelling Cranes.* (18) Dec. 14.
Electricity *versus* Gas for Street Lighting. T. Osborne. (27) Dec. 1.
The Change of Energy Loss with Speed in Direct-Current Machines.* W. M. Thornton. (73) Dec. 20.
The Billiter Alkali-Chlorine Cells.* A. G. Allmand. (Abstract of paper read before the Faraday Soc.) (73) Dec. 20.
The New Wireless Telegraph Station at Fort Myer, Virginia.* Davis H. Tuck and Millard B. Hodgson. (26) Dec. 20.

*Illustrated.

Electrical—(Continued).

Electric Power Testing Set.* James C. Bennett. (16) Dec. 21.
Porcelain-Clad Reactors.* (27) Dec. 21.
Electric Development in New England.* (27) Dec. 28.
Disturbances of Potential and Current Produced in an Active Conducting Network by the Application of a Leak Load.* A. E. Kennelly. (27) Dec. 28.
High Speed Turbo-Alternators—Designs and Limitations. B. G. Lamme. (42) Jan.
Industrial Lighting.* C. L. Eshleman. (42) Jan.
Les Propriétés Magnétiques des Alliages des Métaux Ferromagnétiques; Fer-nickel, Nickel-cobalt, Cobalt-fer. P. Weiss. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
Untersuchungen über magnetische Hysteresis.* F. Holm. (48) Oct. 26.
Einphasenwechselstrom-Kollektormotoren.* M. Latour. (41) Nov. 28.
Selbstkosten von Gasanstalten und Elektrizitätswerken. Friedrich Ross. (41) Nov. 28.
Vereinfachte Berechnung der Spannungsverhältnisse bei Freileitungen mit hohen Spannungen.* Nils Forssblad. (41) Dec. 5.
Ueber Selbsterregung und Nutzbremsung von Maschinen mit Reihenschlusscharakteristik.* A. Scherbius. (41) Dec. 5.
Eine neue künstliche Leitung zur Untersuchung von Telegraphierströmen und Schaltvorgängen.* Karl Willy Wagner. (41) Serial beginning Dec. 12.
Zur Berechnung der mittleren Beleuchtung rechteckiger Flächen.* Adolf Thomälen. (41) Dec. 19.

Marine.

Electrical Working of Auxiliary Machinery on Modern Steamships.* E. T. Caparn. (Abstract.) (77) Nov.
The Uses of Gases on Ships for Fire Extinction and Fumigation. E. Kilburn Scott, A. M. Inst. C. E. (Paper read before the Inst. of Marine Engrs.) (47) Nov. 29.
Performance of a Dipper Dredge Driven by Oil Engines on Drainage Ditch Excavation. (13) Dec. 5.
Modern Painting Methods in the Navy: The Government Now Its Own Manufacturer of Paints. Henry Williams. (Paper presented to the Eighth Inter. Cong. of Applied Chemistry.) (19) Dec. 7.
Use of Oil Fuel in the United States Navy. H. I. Cone. (Abstract of paper read before the Eighth Inter. Cong. of Applied Chemistry.) (13) Dec. 19.
Geared Turbines; Their Development for Ship Propulsion. George Westinghouse. (19) Dec. 21.
Le *Selandia* et le *Jutlandia*, Navires à Moteurs Diesel, de la Compagnie de l'Est-Asiatique Danois. M. Gouriet. (33) Nov. 30.

Mechanical.

On the Measurement of the Air-Supply to Internal Combustion Engines, by Means of a Throttle-Plate. William Watson and Herbert Schofield. (75) Apr.
Furnace Efficiency. Joseph Harrington. (4) Nov.
Purchase of Lubricating Oil by Specification. A. D. Smith. (58) Nov.
Power-Plant Economics.* Horace W. Flashman. (6) Nov.
Coal-Conveying Plant at the Leipzig Gas-Works.* Hubert Hermanns. (66) Nov. 26.
Recent Developments in the Manufacture of Refractory Goods from Fire-Clay. G. H. Pearson-Perry. (Paper read before the Soc. of British Gas Industries.) (66) Nov. 26.
The Measurement of the Flow of Gases in Mains by Means of the Pitot Tube.* A. R. Griggs. (Paper read before the London and Southern District Junior Gas Assoc.) (66) Nov. 26.
Mechanical Haulage. George Symon. (Paper read before the Institution of Mun. Engrs.) (104) Nov. 29.
Some Milling Experiments.* P. V. Vernon. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Nov. 29.
Chain Driving.* H. T. Hildage, M. Inst. C. E. (Paper read before the Rugby Eng. Soc.) (47) Nov. 29.
Examples of Milling Machine Work.* (108) Serial beginning Dec.
Hydraulic Cranes, Immingham Docks; Gt. Central Railway.* (21) Dec.
Laboratory on Wheels; Used in Atmospheric Tests by Smoke Abatement Experts.* Hugh Pattison. (60) Dec.
Manufacture of Railway Shovels.* (87) Dec.
Coal Stacking and Coal Firing. H. Kendrick. (Paper read before the Manchester District Institution of Gas Engrs.) (66) Dec. 3.
High-Pressure Gas Lighting. Walter Grafton. (Paper read before the Scottish Junior Gas Assoc.) (66) Dec. 3.
Methods of Laying Submerged Gas Mains Across Two Rivers at New Haven, Conn. H. E. White. (Paper read before the Connecticut Soc. of C. E.) (86) Dec. 4.

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Mechanical (Continued).

Canadian Studies of Peat Fuel in Gas Producers and the Elimination of Tar from Producer Gas.* (13) Dec. 5.
The Molding of Bronze Statuary.* E. A. Suverkrop. (72) Dec. 5.
Motorcycle Work at the Pierce Shop.* Ethan Viall. (72) Dec. 5.
A New Principle in the Design of High-Vacuum Pumps.* (13) Dec. 5.
English Tower Cranes for Building Construction.* (13) Dec. 5.
Universal Boring and Milling Machine with Internal Spindle.* (13) Dec. 6.
A Powerful Slotting Machine.* (12) Dec. 6.
Note on Fly-Wheel Design for Internal Combustion Engines.* Robert Oliphant Boswall. (13) Dec. 6.
Waste Heat Utilization. I. V. Robinson, M. Inst. Mech. Engrs. (Abstract of paper read before the Cleveland Eng. Soc.) (22) Dec. 6.
The Rating and Pricing of Speed Reducing Gear.* E. A. Vessey. (22) Dec. 6.
Open-Hearth Furnace Design and Manipulation.* John Plochn. (Paper read before the Am. Foundrymen's Assoc.) (47) Dec. 6; (20) Dec. 26.
Mixing, Hoisting and Charging Equipment for Handling Coal and Minerals, Installations for Handling with Automatic Devices 900 Tons of Material a Day.* (14) Dec. 7.
Some Recent Developments in Wood Distillation. Thomas W. Pritchard. (Abstract of paper read before N. Y. Section of Soc. of Chemical Industry.) (19) Dec. 7.
The Barnum Process of Generating Gas.* (24) Dec. 9.
Tests of Wood, Paper and Steel Pulleys.* H. A. Woodworth. (64) Dec. 10.
The Design of Ignition Arches.* Joseph Harrington. (64) Dec. 10.
Small Westinghouse Steam Turbine.* (64) Dec. 10.
Indirect Lighting. F. W. Willcox and H. C. Wheat. (Paper read before the Illuminating Eng. Soc.) (66) Dec. 10.
Vertical Retorts and Continuous Carbonization.* Norton H. Humphreys, Assoc. M. Inst. C. E. (66) Dec. 10.
A Quadrant Crane for Motor Trucks.* (13) Dec. 12.
An Aeroplane Catapult.* (12) Dec. 13; (46) Dec. 14.
Furnace Charging Machines with Special Reference to Open Hearth Works.* J. Smith, A. M. I. Mech. E. (73) Dec. 13.
Pyrometry in Steel Works.* Chas. R. Darling. (73) Dec. 13.
The Manufacture of Seamless Steel Boiler Tubes. J. J. Dunn. (Abstract of paper read before the Am. Boiler Mfrs.' Assoc.) (18) Dec. 14.
How to Overcome Lamination.* (Brick Manufacture.) R. T. Stull. (76) Dec. 15.
Consumers' Meters. Charles C. Schiller. (Paper read before the Am. Gas Inst.) (83) Dec. 16.
Heat Value and Candle Power. (Report of Committee of the Am. Gas. Inst.) (83) Dec. 16.
Some Practical Notes on Retort House Work. Victor White. (24) Dec. 16.
Nashamena Steam Turbine Plant.* Warren O. Rogers. (64) Dec. 17.
The Flow of Steam Through Pipes.* H. V. Carpenter. (64) Dec. 17.
The Residuals Inquiry. (Text of the Report by the Joint Select Committee of the House of Lords and the House of Commons on "Gas Authorities (Residual Products.")) (66) Dec. 17.
Lignite and Its Uses. R. O. Wynne-Roberts. (Paper read before the Regina Eng. Soc.) (96) Dec. 19.
Group and Individual Drives.* A. G. Popcke. (72) Dec. 19.
Stresses and Deflections of Shafts.* A. Schein. (72) Dec. 19.
A Magnetic-Load Machine for Alternating Stress Tension Tests, and Some Results.* (13) Dec. 19.
Aeroplane Engines.* A. Graham Clark. (Paper read before the Institution of Automobile Engrs.) (13) Dec. 20.
The Keymer Rock-Drill.* (13) Dec. 20.
Test-Bars for Chillable Irons.* Thos. D. West. (Paper read before the Inter. Assoc. for Testing Materials.) (13) Dec. 20.
Recent Developments in the Curtis Steam Turbine.* R. F. Halliwell. (Paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Dec. 20.
The Crosby Automatic Boiler-Feed Regulator. (13) Dec. 20.
Flat Surfaces Supported by Stay Bolts.* A. J. Toppin. (64) Dec. 24.
The Low-Temperature Modification of Bituminous Coals to Form a Smokeless Domestic Fuel.* (Abstract from *Bul.*, Univ. Ill. Eng. Exper. Station.) (13) Dec. 26.
The Case of the Oil Engine. William T. Price. (Paper read before the Philadelphia Foundrymen's Assoc.) (20) Dec. 26.
Freight Handling System for a Warehouse, Telpherage System for Handling Detachable Motor Truck Bodies Loaded with Packages of Miscellaneous Sizes.* (14) Dec. 28.
The Manufacture of Manila Rope; Its Use for Transmission and Hoisting.* C. W. Hunt. Dec. 28.
The Photometry of Incandescent Gas Lamps.* C. O. Bond. (Paper read before the Am. Gas Inst.) (24) Dec. 30.

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Mechanical—(Continued).

Lentz System Applied to Steam Engine.* Sigfried Rosenzweig. (Paper read before the Ohio Soc. of Mech. Elec. and Steam Engrs.) (64) Dec. 31.
Progress of Wolf Locomobile Engines.* F. E. Junge. (64) Dec. 31.
Procedure in Coal Specifications. Mervin K. Baer. (64) Dec. 31.
Les Trains Planétaires, leur Théorie et leur Application aux Réducteurs et Multiplicateurs de Vitesse.* P. Laruelle. (37) Nov. 30.
Presses à Agglomérer les Charbons, pour la Fabrication des Boulets Ovoïdes, Construites par les Fonderies et Ateliers de Construction de l'Horme (Loire).* (34) Dec.
Enregistrement Automatique de la Relation entre les Efforts et la Déformation des Matériaux lors du Choc.* André Gagarine. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
Recherches Expérimentales sur les Métaux Antifriction pour Machines.* Nino Pecoraro. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
Application à la Fonte de Méthodes d'Essais Mécaniques Adoptées pour les Autres Métaux.* A. Damour. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
IIe Concours de Pare-Boue de l'Automobile-Club de Seine-et-Oise (Versailles, 1912).* E. Bret. (33) Dec. 14.
Eine Drahtseilbahnanlage von ungewöhnlichen Abmessungen.* Adolf Bleichert. (81) Pt. 6.
Die Herstellung von Qualitätsguss unter Verwendung von Metallspänen.* J. Mehrtens. (48) Oct. 26.
Neuere Bestrebungen im Dampfkesselbau.* Friedrich Münzinger. (48) Serial beginning Oct. 26.
Ueber die Wahl der Geschwindigkeitsdiagramme von Francis-Turbinen.* K. Körner. (48) Oct. 26.
Zur Schleiftechnik in der Giessereibetrieben der Vereinigten Staaten von Nordamerika.* C. Krug. (50) Nov. 28.
Selbstkosten von Gasanstalten und Elektrizitätswerken. Friedrich Ross. (41) Nov. 28.

Metallurgical.

Methods of Preparing Cuban Brown Iron Ore for Blast Furnace Use. R. B. Gerhardt. (98) Nov.
The Black Oak Mill.* Charles H. Urquhart. (103) Nov. 30.
Hydraulic Classification. (Pulp for Tnbe Milling.) G. A. Robertson. (105) Dec.
Recent Copper Milling Practice at Lake Superior.* (105) Dec.
The Function of the Slag in Electric Steel Refining. Richard Amberg. (Paper read before the Inter. Cong. of Applied Chemistry.) (22) Dec. 6.
The Electric Furnace for Brass Melting. G. H. Clamer and Carl Hering. (Abstract of paper read before the Am. Inst. of Metals.) (47) Dec. 6.
The Froth Flotation Process. H. L. Sulman. (Paper read before the Inst. of Mines and Metallurgy.) (16) Dec. 7.
Nipissing High-Grade Mill, Cobalt.* R. B. Watson. (16) Dec. 7.
Iron and Steel Smelting in Electro-Metals Furnaces. T. D. Robertson. (73) Dec. 13.
Nipissing High-Grade Mill, Cobalt. Herbert A. Megraw. (16) Dec. 14.
Mechanical Efficiency in Crushing.* Algernon Del Mar. (16) Dec. 14.
Evolution of an Electrolytic Refinery. Harold French. (103) Serial beginning Dec. 14.
Amalgamation at the Homestake.* Allan J. Clarke and W. J. Sharwood. (103) Dec. 14.
Progress in Steel Making in Alabama. Frank H. Crockard. (From Alabama Geol. Survey Publication.) (20) Dec. 19.
Induction Furnaces for Steel Refining. John B. C. Kershaw. (12) Serial beginning Dec. 20.
The Open Hearth Furnace for Malleable Cast Iron. G. A. Blume. (Abstract of paper read before the Am. Foundrymen's Assoc.) (22) Dec. 20.
Hollinger Cyanide Mill, Porcupine.* Herbert A. Megraw. (16) Dec. 21.
Cyanide Practice in the Black Hills.* Herbert A. Megraw. (16) Dec. 28.
Rapport sur les Progrès de la Metallographie depuis la Commencement de l'Année 1909 jusqu'à la Fin de 1911.* E. Heyn. (Report of Comm. to Inter. Assoc. for Testing Materials.) (93) Dec.
Les "Inclusions de Scories." Walter Rosenhain. (Report to Inter. Assoc. for Testing Materials.) (93) Dec.
Principes Généraux pour les Conditions de Réception du Cuivre. Léon Guillet. (Report of Comm. to Inter. Assoc. for Testing Materials.) (93) Dec.
De la Soudure des Soufflures et Cavités du Lingot d'Acier.* J. E. Stead. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
La Biographie de la Cémentite Pro-Eutectoïde.* Henry M. Howe et Arthur G. Levy. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.

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Metallurgical (Continued).

Note sur la Croissance Cristalline de la Ferrite Au-Dessous de sa Zone Critique de Température.* Albert Sauveur. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
La Désintégration Electrique des Métaux et son Emploi Possible pour les Essais.* Carl Benedicks. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
Les Impuretés Solides Non-Métalliques de l'Acier Sonims.* Henry D. Hibbard. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
Etudes Micrographiques, Mémoire sur Quelques Observations Micrographiques Ayant Amené des Résultats Pratiques, dans la Réception, l'Examen ou le Mode de Travail de Différents Métaux.* (Paper read before the Inter. Assoc. for Testing Materials.) (93) Dec.
Ueber den Einfluss des Arsens auf die Eigenschaften des Flusseisens.* J. Liedgens. (50) Dec. 19.

Military.

The Military Supremacy of the Air: The Aeronautic Plans of Great Military Powers. Theodore M. R. von Kéler. (46) Serial beginning Dec. 28.

Mining.

The New Rand Goldfield.* A. R. Sawyer. (Paper read before the North Staffordshire Inst. of Min. and Mech. Engrs.) (106) Vol. 44, Pt. 1.
The Illumination at the Coal-Face with Special Reference to the Incidence of Miners' Nystagmus. T. Lister Llewellyn. (Paper read before the South Staffordshire and Warwickshire Inst. of Min. Engrs.) (106) Vol. 44, Pt. 1.
Maltby Main Colliery. Alfred Thompson. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs. and the Midland Counties Institution of Engrs.) (106) Vol. 44, Pt. 1.
Some Experiences with Winding-Ropes and Capels. W. D. Lloyd. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 44, Pt. 1.
Facts and Theories Relating to Fans. David M. Mowat. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 44, Pt. 1.
The Ignition of Coal-Gas and Methane by Momentary Electric Arcs.* W. M. Thornton. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 44, Pt. 1.
Electrically-Driven Winding-Engines in South Africa.* A. W. Brown. (106) Vol. 44, Pt. 1.
The Use of Old Wire Ropes in Timbering Roadways. John McLuckie. (Paper read before the Min. Inst. of Scotland.) (106) Vol. 44, Pt. 1.
The Effects of Deficiency of Oxygen on the Light of a Safety-Lamp. J. S. Haldane and T. Lister Llewellyn. (Paper read before the South Staffordshire and Warwickshire Inst. of Min. Engrs.) (106) Vol. 44, Pt. 1.
The Energy of Explosives.* Walter O. Snelling. (58) Oct.
The Chuichos Coal Mine, Peru. Lester W. Strauss. (6) Nov.
Notes on Diamond Drilling. Albert E. Hall. (6) Nov.
The Electrification of a Group of Small Collieries. Campbell King. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Nov. 29.
The Mount Lyell Disaster.* Hartwell Conder. (68) Nov. 30.
The Production of Available Potash from the Natural Silicites. Allerton S. Cushman and George W. Coggeshall. (3) Dec.
Application of Concrete to Underground Works.* H. T. Mercer. (Paper read before the Lake Superior Min. Inst.) (96) Dec. 5.
Large Synchronous Motors for Compressor Service (Mining Operations).* Girard B. Rosenblatt. (103) Dec. 7.
Marquette Range Shows Continued Progress.* (82) Dec. 7.
Electricity in Metal Mining in Colorado.* W. J. Canada. (27) Dec. 7.
The Santa Maria Graphite Mines, Mexico.* W. D. Hornaday. (82) Dec. 7.
Insect Damage to Mine Props and Its Prevention. T. E. Snyder. (82) Dec. 7.
Methods and Cost of Drilling in the California Oilfields.* A. T. Parsons. (86) Dec. 11.
100-Horse-Power Electrically Driven Hauling Gear.* (12) Dec. 13.
Recording the Ventilation in Mines.* (22) Dec. 13.
The Wellesley Colliery of the Wemyss Coal Co., Ltd.* Jas. D. Welch. (Paper read before the Scottish Federated Inst. of Min. Students.) (22) Dec. 13.
Square-Set Timbering: Importance and Evolution. Claude T. Rice. (82) Serial beginning Dec. 14.
Application of Electric Power for Coal Mining.* S. R. Stone. (82) Dec. 14.
The Treasury Tunnel Raise. H. T. Russel. (16) Dec. 14.
Mine Slope Economizing Hand Labor.* John J. Smith. (16) Dec. 21.
No. 5 Tunnel, Mammoth Mine, California.* Robert E. Hanley. (16) Dec. 21.
Mining Methods in the Waihi Mine.* Jas. L. Gilmour and W. H. Johnston. (From *Proceedings*, Australasian Inst. of Min. Engrs.) (103) Dec. 21.
The Federal Government and Mineral Lands. W. C. Mendenhall. (82) Dec. 21
Obtaining Efficiency in Mining. Andre Formis. (16) Dec. 28.

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Miscellaneous.

The Manufacture of Sugar from Wood, and its Economic Importance. A. Zimmermann. (29) Dec. 6.
The Quartz Mercury Vapour Lamp.* (22) Dec. 6.
Two Conflicting Theories of Valuation of Public Service Companies. Halbert P. Gillette. (86) Dec. 11.
A German Code of Fees. (13) Dec. 12.
Natural and Synthetic Rubber. F. Mollwo Perkin. (29) Dec. 13.
The Parcels Post Zone System.* Emma M. V. Triepel. (46) Dec. 14.
Die Dampkraft und andere Energiequellen im zukünftigen Transportwesen.* Leopold Kliment. (53) Serial beginning Dec. 6.

Municipal.

Temperature Strains in Brick Pavements.* James E. Howard. (60) Dec.
Methods and Cost of Roadway Reconstruction in the Lincoln Park System, Chicago, during 1911.* M. M. Lawrence. (86) Dec. 4.
Method and Cost of Levelling Dredged-Over Ground with Electrically Driven Drag Scraper.* James C. Bennett. (86) Dec. 4.
Brantford Concrete Pavements.* T. Harry Jones, M. Can. Soc. C. E. (96) Dec. 5.
Tests of Road Stones in the United States. (From *Bulletin*, U. S. Office of Public Roads.) (104) Dec. 6.
Sand-Clay Roads in North Carolina. (14) Dec. 7.
The Splinter Treatment and Waste Sulphite Liquor Oil Sand Treatment as Practiced in Connecticut. Maurice O. Eldridge, Assoc. M. Am. Soc. C. E. (86) Dec. 11.
Plant Equipment for Road Construction. F. E. Ellis. (Paper read before the Am. Good Roads Congress.) (86) Dec. 11.
Maintenance and Repair of New York State Highways in 1911. (From Annual Report, N. Y. State Comm. of Highways.) (96) Dec. 12.
Road Building at a Mile-per-Day Rate. G. Howland Leavitt. (Abstract of paper read before the Am. Road Builders' Assoc.) (14) Dec. 14.
Town Planning from an Engineering Aspect. Ernest R. Matthews. (Abstract of paper read before the Soc. of Engrs.) (14) Dec. 14.
Some Features of Macadam Construction. T. R. Agg. (Abstract of paper read before the Am. Road Builders' Assoc.) (14) Dec. 14.
Wood Block Pavement Laid by City Labor. Ellis R. Dutton. (Abstract of paper read before the Am. Road Builders' Assoc.) (14) Dec. 14; (60) Dec.
Bituminous Pavements for City Streets. George W. Tillson. (Abstract of paper read before the Am. Road Builders' Assoc.) (14) Dec. 14.
Methods and Costs of Street Cleaning at Washington, D. C., during 1911-12. (86) Dec. 18.
The Resurfacing of 102 Miles of Highway in Five Months. (86) Dec. 18.
Experience with Bituminous Roads and Road Treatment. William B. Sohier. (Abstract of paper read before the Am. Road Builders' Assoc.) (14) Dec. 21.
Proposed Traffic Regulation on Fifth Avenue, New York. (14) Dec. 21.
Methods and Labor Costs of Concrete Pavement Construction at Davenport, Ia., by Hand Mixing and by Machine Mixing.* W. S. Anderson. (86) Dec. 25.
Town Planning and Civic Improvement. C. H. Mitchell. (96) Dec. 26.
A Successful Budget-Method Protest, Illustrated by Extracts from the Report of the Bureau of Street Cleaning, Richmond Borough, New York City. (13) Dec. 26.
Concrete Construction of the Watertown Plank Road, Wisconsin.* H. J. Kuelling. (13) Dec. 26.
Concrete Pavements Tamped with Mechanical Vibrator. (14) Dec. 28.
Nouveau Système Economique de Rechargement des Chaussées Empierrées. A. Sallé. (35) Sept.
Strassenteerungen.* Hentrich. (39) Nov. 20.
Die Abwässer der Fabriken als Strassenstaubbekämpfungsmittel. (39) Nov. 20.
Das Automobil auf der Landstrasse und die Staubplage. Metzmacher. (39) Nov. 20.
Die Radrennbahn Zürich-Oerlikon.* M. Scheifele. (107) Serial beginning Dec. 7.

Railroads.

Headlights Tests. C. M. Larson. (61) Oct. 15.
Mechanical Transferage at Railway Shops and Terminals.* H. McL. Harding. (61) Nov.
Electrification of the Melbourne Railways. Charles Merz. (Report presented to the Government.) (12) Nov. 29.
Mallet Articulated Compound Locomotive (2-8-8-0 Type), G. N. R., U. S. A.* (11) Nov. 29.
Sugden's Superheater.* (11) Nov. 29.
Electrification of Main Lines. G. Brecht. (Translated from *Elektrische Kraftbetriebe*.) (73) Serial beginning Nov. 29.
Railroad Valuation: Reproduction Cost New as a Sole Basis for Rates. D. F. Jurgensen. (Paper read before the Civ. Engrs'. Soc. of St. Paul.) (1) Dec.

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Railroads—(Continued).

Concrete Practice, No. 5, C. C. & St. L. Ry. Co.* A. M. Wolf. (87) Dec.
Standard Signal Practice on the L. & N.* B. W. Meisel. (87) Dec.
2-6-0 Express Goods Engines, Great Northern Railway.* (21) Dec.
Bogie Carriages for the Chilian Longitudinal Railway—Northern Section.* (21) Dec.
New Locomotives, Northern of France Railway.* (21) Dec.
Bulgarian Railway Locomotives.* (21) Dec.
Watervliet Terminal, Delaware & Hudson Co.* J. C. Chapple. (87) Dec.
Northumberland Yard, Northern Central Ry.* (87) Dec.
Temperature Tests on Superheater Locomotives.* (25) Dec.
Largest Narrow Gage Locomotives.* (25) Dec.
The Progress in Testing Full-Size Pieces Under Practical Conditions, Together with Locomotive Testing in the United States. Gaetano Lanza. (3) Dec.
A High-Speed German Compound Locomotive with Schmidt Superheater.* Frank C. Perkins. (94) Dec.
Notes on the Working of Coupling Rods.* G. L'Hoest. (88) Dec.
Electrically-Operated Suspension Lines in Goods Sheds.* Fenten. (From *Zeitung des Vereins deutscher Eisenbahnverwaltungen.*) (88) Dec.
The Locomotive Superheater and Some of Its Effects on the Cost of Railway Operation.* Gilbert E. Ryder. (65) Dec.
The Construction of a West African Railway.* G. M. Harris. (Paper read before the Inst. of Civil Engrs. of Ireland.) (96) Dec. 5.
State Railroads of the Dutch East Indies. D. C. Alexander, Jr. (Abstract from *Daily Consular and Trade Reports.*) (20) Dec. 5.
Rail Anchors or Anti-Creepers.* (13) Dec. 5.
Lana-Vigiljoch Aerial Cable-Way.* (12) Dec. 6.
Derailment at Glen Loch, Pa.* (15) Dec. 6.
The Hopatcong-Slateford Cut-Off.* C. W. Simpson. (15) Dec. 6.
A Gateway to the Heart of New York: The New Grand Central Station and Its Relation to New York Traffic.* (19) Dec. 7; (46) Dec. 7; (27) Dec. 21.
The Optics of a Railway Signal Lamp.* T. A. Lawes. (19) Dec. 7.
Methods of Promoting Efficiency in Maintenance on the Pittsburgh and Lake Erie.* (14) Dec. 7.
Driving and Lining the New Sand Patch Tunnel.* (14) Dec. 7.
Pacific and Mikado Type Locomotives for the New Orleans, Mobile and Chicago R. R.* (18) Dec. 7.
Mountain Cableway for Passengers in Austria.* Francis P. Mann. (14) Dec. 7.
The Windsor, Essex and Lake Shore Rapid Railway.* (96) Dec. 12.
Rail Corrugations.* H. T. Wakelam. (Paper read before the Institution of Mun. and County Engrs.) (104) Dec. 13.
Cause of the Glen Loch Derailment. (15) Dec. 13.
Efficient Organization for Handling Salvage.* H. C. Pearce. (15) Dec. 13.
Repair Shop of the New York, Westchester & Boston Railway.* (17) Dec. 14.
Railroad Rates and Rate Making. O. P. Gothlin. (Abstract from Report of the Public Service Comm. of Ohio.) (18) Dec. 14.
Headlight Tests, Railroad Commission of Wisconsin. (18) Dec. 14.
Electrification of the London and North-Western Suburban Lines.* (12) Dec. 20
Tunnel Memorials.* (12) Dec. 20.
Locomotive Feed Water Heating.* (12) Dec. 20.
The Country's Need of Greater Railroad Facilities. James J. Hill. (Abstract of paper read before the Ry. Business Assoc.) (15) Dec. 20; (18) Dec. 21.
Chrome-Vanadium Driving Wheel Tires.* (15) Dec. 20.
Prolonging the Life of Crossties.* Howard F. Weiss. (From *Bulletin 118*, U. S. Forest Service.) (15) Dec. 20.
Keeping Stone Ballast Clean. C. E. Lindsay. (15) Dec. 20.
Solution of the Steel Rail Problem.* (46) Dec. 21.
The Westport Disaster: Report of the Connecticut Public Utilities Commission.* (19) Dec. 21; (15) Dec. 13; (18) Dec. 14.
Electric Express Service at Boston.* (17) Dec. 21.
Grade Revision and Double-Tracking on the A. T. & S. F. Ry.* J. G. Van Zandt. (18) Dec. 21.
The Glen Loch Wreck, More Facts.* Charles A. Wentworth. (13) Dec. 26.
An Automatic Stop Experiment on the Lackawanna. (15) Dec. 27.
Engineering and Operating Features of the Tri-City Railway, Davenport, Ia.* (17) Dec. 28.
Oil-Burning Locomotives for the Baltimore & Ohio R. R.* (18) Dec. 28.
Specifications for Heat-Treated Chrome Vanadium Steel Tires.* (18) Dec. 28.
Six-Tracking a Heavy-Traffic Line and Elevating for Grade-Crossing Elimination, Pennsylvania Railroad Improvement at Rahway, N. J.* (14) Dec. 28.
Souterrain de la Gare de Paris-Nord pour le Transport des Bagages. Sabourin et Théry. (38) Oct.
Note sur les Nouvelles Voitures de la Cie. de Bône-Guelma et Prolongements (Réseau à Voie de 1m.).* Félix. (38) Oct.
Les Chemins de Fer du Massif du Mont-Blanc, le Funiculaire aerien de l'Aiguille du Midi.* P. Dalimier. (33) Nov. 23.

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Railroads—(Continued).

Le Métropolitain Souterrain à Marchandises de Chicago.* (33) Nov. 30.
Nouveau Dépot de Locomotives de Mohon.* Henry et Valat. (38) Dec.
Locomotive-Tender à Six Essieux Couplés et à Surchauffe, des Chemins de Fer de Java.* (33) Dec. 14.
Bearbeitung der Heizrohre der Lokomotiven in der Werkstätte Pilsen.* Karl Kramár. (102) Nov. 15.
Achsbüchse von Cosmovici.* (102) Nov. 15.
Ablaufanlagen auf Verschiebebahnhöfen für reinen Schwerkraftsbetrieb.* Sammet. (102) Serial beginning Nov. 15.
Die erste mit Zwischenstützen versehene Personen-Bergseilschwebebahn deutscher Bauart.* M. Buhle. (40) Nov. 23.
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PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

HYDROLOGY OF THE PANAMA CANAL.

By Caleb Mills Saville, M. Am. Soc. C. E.*

To be Presented March 5th, 1913.

A most critical and interesting period in the history of the discharge from the Chagres River Basin has just been completed, and, on account of the wide-spread interest in the Panama Canal, it seems pertinent just now to put on record certain data bearing on the hydrology and other natural phenomena of this basin. In comparatively close sequence have followed the year of minimum run-off, the year of least dry-season run-off, and the year of maximum discharge, and these and other facts relating to the water supply of the lake seem to be of especial interest at this time. In a way, this paper may be considered as an extension of the records set forth previously by A. P. Davis, M. Am. Soc. C. E.,† in his reports,‡ and by Gen. H. L. Abbot, Engineer Corps, U. S. A. (Retired), in his several monographs on this subject. Especial attention is directed to "Problems of the Panama Canal," by the latter author, in which may be found valuable compilations of former records, some of which are not available elsewhere.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Formerly Assistant Engineer in Charge of Third Division, Office of Chief Engineer, Isthmian Canal Commission.

† Chief Engineer, U. S. Reclamation Service.

‡ "Hydrology of the American Isthmus," and "Hydrology of Nicaragua." (U. S. Pub Doc.)

These and the deductions therefrom are the careful work of a trained hydraulician, and represent painstaking investigation.

The data and records used in the preparation of this paper were taken from the hydrographic and meteorological files of the Isthmian Canal Commission by the kind permission of Col. George W. Goethals, Corps of Engineers, U. S. A., M. Am. Soc. C. E., Chairman and Chief Engineer of the Isthmian Canal Commission.

Introduction.

As is generally known, the Isthmian Canal is being constructed across the Isthmus of Panama from the Caribbean Sea to the Gulf of Panama, and its location is approximately in the center of a strip of land, 10 miles wide, the use and control of which was granted in perpetuity to the United States by the Republic of Panama. This strip of land, called the "Canal Zone," stretches in a generally northeast and southwest direction, and is approximately contained between Longitude 79° 30′ and 80° 00′ west and Latitude 8° 55′ and 9° 25′ north.

Topography.

Most of that portion of the Isthmus of Panama which lies in the immediate vicinity of the Canal Zone is of little elevation above sea level, and the higher portions are either a series of more or less parallel ridges of nearly uniform height, or isolated dome-shaped hills, seemingly dropped haphazard. Along the Caribbean Coast there are many stretches of fresh swamp land, such as the Mindi Marshes southwest of Colon and reaching inland 6 or 7 miles. On both sides of the Isthmus there are also extensive salt-water shallows, mostly covered with low-growing mangroves. On the other hand, there are many bold bluffs of hard trap rock which rise abruptly from the sea, and ridges of these run back into the higher hills inland. The harbor of Porto Bello is nearly surrounded by such a barrier, while at Limon Bay, the harbor of Colon, the opposite condition appears.

On the Pacific side similar shore conditions are to be found, except that, for miles along the coast in each direction, there are no harbors which can compare with those on the Atlantic side. The main drainage of the Isthmus, of course, is dependent on the cordillera, or continental divide. About 4 miles southeast of Panama City the divide approaches within 7 miles of the Pacific shore, and for the entire dis-

tance along the southerly boundary of the Chagres Basin it is nowhere much more than 11 miles from this ocean.

The Chagres River is the largest in the Republic of Panama on the Atlantic side of the Isthmus. It has a drainage area above Gatun of 1 320 sq. miles, and between this station and the mouth of the river there are probably about 25 sq. miles of additional drainage. The Chagres discharges into the Caribbean Sea about 5 miles southwest of Toro Point, which is the northernmost limit of Canal Zone land on the westerly side of Limon Bay. Going up stream from its mouth, the course of the river is generally southeastward to Gamboa, which is about half way across the Isthmus and 35 miles from the mouth of the river. Up to this point, the rise has been very slight, the elevation of the river bed at Gamboa being only 42 ft. above sea level. The course has thus far meandered through comparatively level and oftentimes swampy bottom lands, which vary in width, to the low hills on each side. The widest stretch of this country is found in the Gatun and Trinidad marshes, lying between Gatun and Ahorca Lagarto, a swampy area about 7 miles long and from 2 to 4 miles wide, with many low, isolated hillocks scattered here and there, rising slightly above swamp level. Near Gamboa the river makes an abrupt turn almost at right angles, and the course is now generally northeastward. Its character also changes at this point, and instead of the sluggish river running through a comparatively level flood plain, with a slope of about 1¼ ft. per mile, there is now a stream, for the most part swiftly running, between banks which are the slopes of the higher hills. The Zone line is passed 40 miles above the mouth and about 5 miles above Gamboa, and the remainder of the course is in the Republic of Panama. At Alhajuela, 11 miles above Gamboa, 38½ miles above Gatun, and 46 miles above the river mouth, the bed of the stream is about 92 ft. above sea level. Twenty-four distinct rapids have been passed, and the rise has been about 50 ft. since leaving Gamboa, the average slope being about 4.6 ft. per mile between these stations. This point is approximately at proposed high-water level in Gatun Lake at its final elevation.

The climate of the Canal Zone, in a general way, has the characteristics stated by Dr. W. F. R. Phillips as belonging to the tropics:

"Unusually mild, equable, moist, warm; average temperature, 80 F.; rainfall frequent and heavy over water and over windward land

exposure; nights unusually clear; afternoons cloudy; no general storms; seasons, rainy and dry, but this division only a relative one."

In regard to climatic changes and the length of time necessary for a complete cycle of fluctuation and evidence of periodicity, Willis J. Moore,* quoting Brückner, states that such a period is completed in an average of 35 years, with possible extremes of 15 years in individual cycles. Dr. W. J. S. Lockyer came to the same conclusion, using different premises from Brückner. The late George W. Rafter, M. Am. Soc. C. E.,† quoting Mr. A. R. Binnie, states that "dependence can be placed on any good record of thirty-five years' duration to give a mean rainfall correct within 2 per cent. of the truth." Mr. Rafter further states that:

"for records from twenty years to thirty-five years in length, the error may be expected to vary from 3.25 per cent. down to 2 per cent., and that for the shorter periods, of five, ten, and fifteen years, the probable extreme deviation from the mean would be 15 per cent., 8.25 per cent., and 4.75 per cent., respectively."

Station Instrumental Equipment.

Three first-class meteorological stations are in operation in the Canal Zone: Ancon, on the Pacific Coast; Culebra, in the interior on the continental divide; and Cristobal, on the Atlantic Coast. Each of these stations is equipped with the following instruments:

Standard.—Mercurial barometer, hygrometer, anemoscope, anemometer, sunshine recorder, automatic rain gauge, maximum, minimum, and standard thermometers.

Self-recording. — Thermograph, hygrograph, barograph, and meteorograph, the latter recording sunshine, rainfall, and direction and velocity of the wind.

The meteorological instruments are all of standard types, similar to those in use by the United States Weather Bureau. The Culebra station is also equipped with a maximum solar thermometer for recording the direct sun temperature, and the two coast stations have water thermographs for recording automatically the temperature of the sea water. Besides the regular meteorological stations, three wind-

* "Descriptive Meteorology."

† "Water Supply and Irrigation Paper No. 80."

movement stations are maintained, at Pedro Miguel, Gamboa, and Gatun. These stations are equipped with anemometer, anemoscope, and meteorograph for recording the direction and velocity of the wind.

Twenty-five rainfall stations are maintained in the Canal Zone, and rainfall records are also received from Chepo, Chorrera, and Bocas del Toro, R. P., and from three other stations outside the Zone and at the head-waters of representative streams.

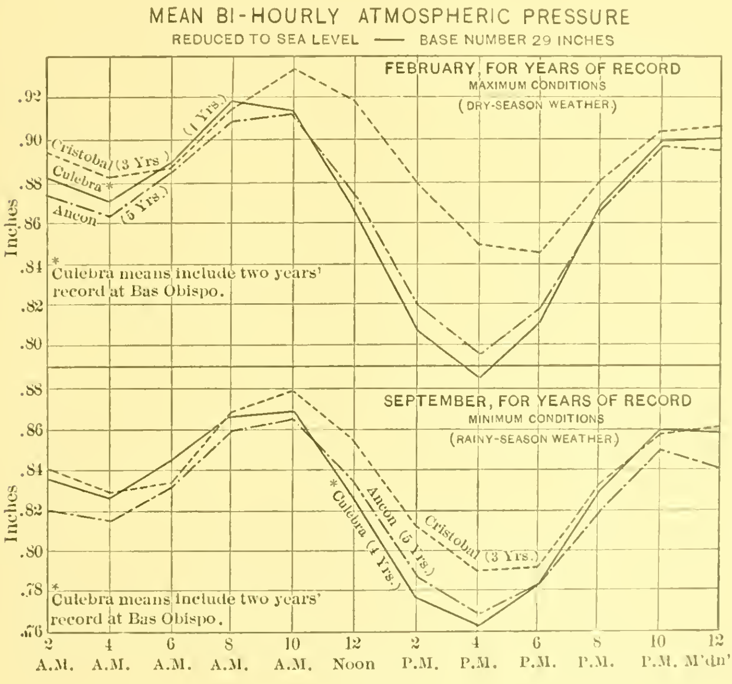


Fig. 1.

Evaporation stations are maintained at Ancon, Rio Grande, and Brazos Brook Reservoirs, and at three selected locations on Gatun Lake, and seismological stations at Ancon and Gatun. Continuous tidal records are kept at Balboa and Cristobal.

For purposes of comparison, Table 1 shows meteorological data from the three first-class stations in the Canal Zone and from Government weather reports of stations in the United States and the Philippine Islands.

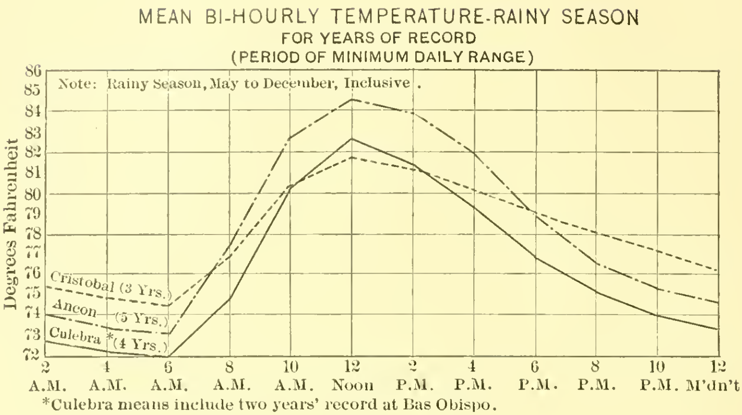

MEAN BI-HOURLY TEMPERATURE-RAINY SEASON
FOR YEARS OF RECORD
(PERIOD OF MINIMUM DAILY RANGE)
Note: Rainy Season, May to December, Inclusive.
Degrees Fahrenheit
86
85
84
83
82
81
80
79
78
77
76
75
74
73
72
Cristobal (3 Yrs.)
Ancon (5 Yrs.)
Culebra * (4 Yrs.)
2 A.M.
4 A.M.
6 A.M.
8 A.M.
10 A.M.
12 Noon
2 P.M.
4 P.M.
6 P.M.
8 P.M.
10 P.M.
12 M'dn't
*Culebra means include two years' record at Bas Obispo.

FIG. 2.

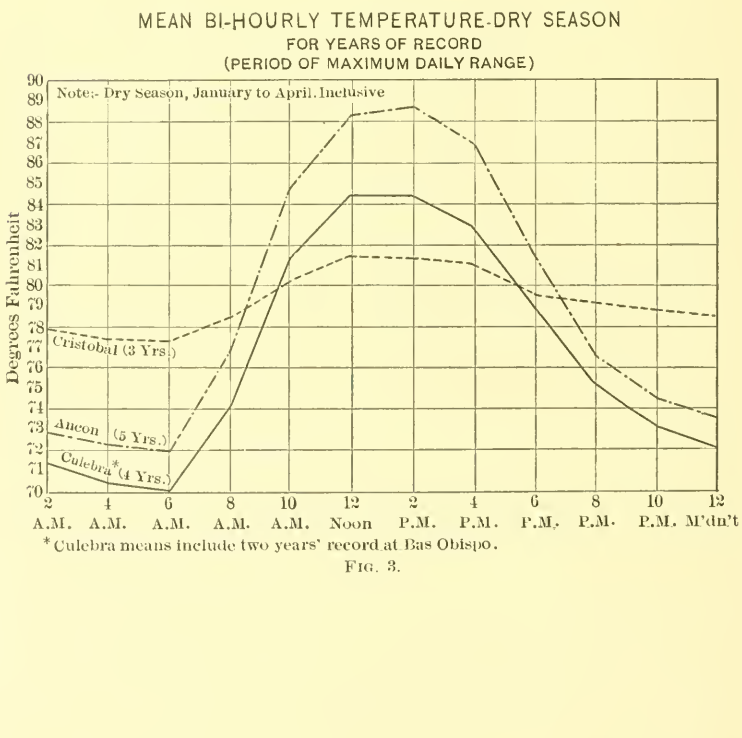

MEAN BI-HOURLY TEMPERATURE-DRY SEASON
FOR YEARS OF RECORD
(PERIOD OF MAXIMUM DAILY RANGE)
Note:- Dry Season, January to April. Inclusive
Degrees Fahrenheit
90
89
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
73
72
71
70
Cristobal (3 Yrs.)
Ancon (5 Yrs.)
Culebra * (4 Yrs.)
2 A.M.
4 A.M.
6 A.M.
8 A.M.
10 A.M.
12 Noon
2 P.M.
4 P.M.
6 P.M.
8 P.M.
10 P.M.
12 M'dn't
* Culebra means include two years' record at Bas Obispo.

FIG. 3.

TABLE 1.—METEOROLOGICAL CONDITIONS AT SELECTED STATIONS.

Station.	Annual rainfall, in inches.	Average number of rainy days per annum.	Mean Relative Humidity. Percentage.			Mean Temperature, in Degrees Fahrenheit.			Mean Wind Movement. Average Hourly.			Extreme Temperature, in Degrees Fahrenheit.	
			Monthly.		Annual.	Monthly.		Annual.	Monthly.		Annual.		
			Max.	Min.		Max.	Min.		Max.	Min.		Max.	Min.
Boston, Mass.	44.70	130	77.0	66.0	72.0	71.3	27.0	48.8	....	....	....	....	...
Philadelphia, Pa.	40.88	129	74.0	64.0	70.0	75.8	31.8	53.6	12.0	8.1	10.3	103	—
Norfolk, Va.	49.29	130	79.0	75.0	78.0	78.0	40.0	58.5	....	....	....	102	....
Jacksonville, Fla.	52.53	126	82.0	73.0	79.5	83.5	73.0	79.5	9.0	7.0	8.0	101	+ 8
Knoxville, Tenn.	48.35	135	72.0	56.0	65.0	76.2	37.5	57.4	8.0	4.7	6.2	100	— 16
Cincinnati, Ohio	40.91	134	76.5	62.0	69.0	77.6	32.9	55.2	8.0	5.0	7.0	105	— 17
Cleveland, Ohio	38.10	159	77.5	70.0	73.5	71.9	27.1	49.3	14.0	9.0	11.0	96	— 18
Marquette, Mich.	32.42	160	82.0	71.0	77.0	65.0	16.7	49.9	11.0	8.0	10.0	108	— 27
Peoria, Ill.	34.75	99	81.5	67.0	75.0	75.4	93.1	49.9	10.8	5.8	8.6	106	— 27
St. Paul, Minn.	27.80	114	80.0	65.0	72.5	72.0	11.9	43.9	8.8	7.1	8.1	104	— 41
Leavenworth, Kans.	34.80	65	....	....	....	78.0	25.4	53.5	....	....	....	115	— 32
Williston, Idaho	14.86	93	77.5	60.5	69.0	68.6	6.1	39.2	11.0	8.0	9.0	107	— 49
Helena, Mont.	13.21	97	69.0	45.5	57.5	66.9	20.0	43.6	....	....	....	103	— 12
New Orleans, La.	55.63	122	79.0	74.5	72.5	81.6	51.1	63.3	8.8	6.1	7.9	102	+ 7
Fort Sill, Okla.	30.85	53	73.5	67.0	70.5	39.5	81.1	60.8	13.0	9.0	11.0	109	— 14
Galveston, Tex.	46.31	104	85.5	76.0	82.5	53.6	83.3	69.8	12.0	8.0	10.0	98	+ 8
El Paso, Tex.	9.31	47	48.0	24.0	39.5	80.6	44.1	62.9	14.0	17.0	10.0	113	+ 5
Prescott, Ariz.	17.40	47	75.0	52.0	62.0	72.7	35.1	52.7	10.0	5.0	7.0	104	— 12
Denver, Colo.	14.05	80	52.0	44.0	50.0	71.8	29.6	49.7	....	....	....	105	— 29
Salt Lake City, Utah	16.33	89	71.0	39.0	52.0	75.5	29.0	51.4	7.0	4.0	6.0	102	— 20
San Diego, Cal.	9.56	39	80.0	73.0	77.0	68.7	54.0	60.6	....	....	....	101	....
San Francisco, Cal.	22.83	71	86.0	77.5	80.5	59.3	51.0	54.9	13.5	6.9	9.7	100	+ 29
Portland, Ore.	42.45	164	85.0	64.0	74.0	66.6	38.7	52.8	7.0	5.0	6.0	102	— 2
CANAL ZONE.													
Ancon, (8° 57′ 25″ N.)	71.67	190	87.6	73.7	82.9	81.8	78.8	80.0	10.0	5.7	7.3	96	63
Culebra, (9° 02′ 23″ N.)	90.95	223	90.0	76.7	85.4	79.0	76.9	78.0	7.3	4.2	5.1	94	59
Cristobal, (9° 21′ 55″ N.)	130.03	273	90.0	79.1	85.6	79.9	77.2	78.5	15.4	5.7	9.6	92	66
PHILIPPINE ISLANDS.													
Manila, (14° 05′ N.)	76.30	....	....	....	79.5	....	....	80.2	....	....	5.7	....	....
Suriago, (9° 48′ N.)	127.86	....	....	....	85.2	....	....	79.6	....	....	....	....	...

TABLE 2.—SUMMARY OF METEOROLOGICAL DATA FOR THE CANAL ZONE FOR THE YEARS OF RECORD.

	Ancon.	Culebra.	Cristobal.
Average number of rainy days	190	223	273
Mean annual rainfall (inches)	71.67	90.95	130.03
Maximum annual rainfall (inches)	91.42 (1901)	107.04 (1896)	183.41 (1909)
Minimum annual rainfall (inches)	45.58 (1882)	64.28 (1888)	86.51 (1884)

Annual rainfall—maximum of record on the Isthmus	237.28 Porto Bello, 1909.
Monthly rainfall—maximum of record on the Isthmus	58.17 " " December, 1909.
	45.03 " " November, 1909.
	42.50 Cristobal, November, 1909.

	Ancon.	Culebra.	Cristobal.
Mean relative humidity (Percentage)			
Maximum month*	87.6 (November)	90.0 (November)	90.0 (November)
Minimum month*	73.7 (March)	76.7 (March)	79.1 (March)
Annual mean*	82.9	85.4	85.6
Mean air temperature (Degrees Fahrenheit).			
Maximum month	81.8 (April)	79.0 (April)	79.9 (April)
Minimum month	78.8 (November)	76.9 (January)	77.2 (November)
Annual mean	80.0	78.0	78.5
Mean hourly wind movement (Miles)			
Maximum month	10.0 (March)	7.3 (April)	15.4 (February)
Minimum month	5.7 (September)	4.2 (September)	5.7 (September)
Annual mean	7.3	5.4	9.6
Extreme air temperature (Degrees Fahrenheit)			
Maximum	96.0 (March 20th, 1908)	94.0 (April 15th, 1909)	92.0 (June 3d, 1909)
Minimum	63.0 (January 27th, 1910)	59.0 (February 9th, 1907)	66.0 (December 3d, 1909)

* Mean of bi-hourly values.
Rainfall records cover 13 years at Ancon, 20 years at Culebra, and 40 years at Cristobal.
Humidity, temperature and wind records cover 5 years at Ancon, 4 years at Culebra, and 3 years at Cristobal.
Culebra records include 2 years at Bas Obispo.

Evaporation.

Evaporation is the name of the process by which aqueous vapor is taken up from a water surface and returned to the atmosphere. By condensation this vapor is changed back into water, and by precipitation the water is again returned to the earth, completing the so-called "meteorological cycle." The prevailing winds, constantly blowing over large bodies of water, keep the saturated vapor always moving on, and dry air behind it flows in, taking up in turn its load of moisture which it carries to land. When the elevation of the land is only a little above water level, there is formed a vast blanket of air thoroughly saturated and ready for any disturbance to cause it to drop its burden in the form of rain. As the air moves away from the water surface, it meets with more and more obstructions, in the form of hills and ridges, and by them is deflected upward into the colder areas. There, becoming condensed, the vapor falls to earth in the form of rain. In the regions of steadily prevailing winds like the trades, the windward slope of the mountain ranges is the one of more copious rainfall.

The laws of evaporation are probably very complex, and although much study and investigation have been given to the phenomena, no satisfactory conclusions have been reached as yet which are applicable to all conditions. The principal factors influencing evaporation are wind, temperature, and vapor pressure. In a region like the Canal Zone, where atmospheric temperature and pressure are exceedingly uniform, the effect of the wind is of paramount importance in causing variation in the rate of evaporation. The diagram, Fig. 5, shows the relation between these factors for the years of record at the Brazos Brook station. The parallelism of the evaporation and wind velocity curves is especially noticeable, as is also the inverse relationship of these curves and the lines of monthly rainfall. The air temperature curve, which is that of the monthly mean, although very uniform, still indicates a certain parallelism with the evaporation curve, while the opposite is shown in the curve of vapor pressure. The diagram, Fig. 8, shows graphically the number of days on which the evaporation at Rio Grande Reservoir and at Brazos Brook Reservoir have exceeded a stated quantity. From these curves it appears that the mean daily evaporation for dry-season conditions is about $\frac{1}{6}$ in. per day, and that this rate is only exceeded for about 80 days. This rate means a lower-

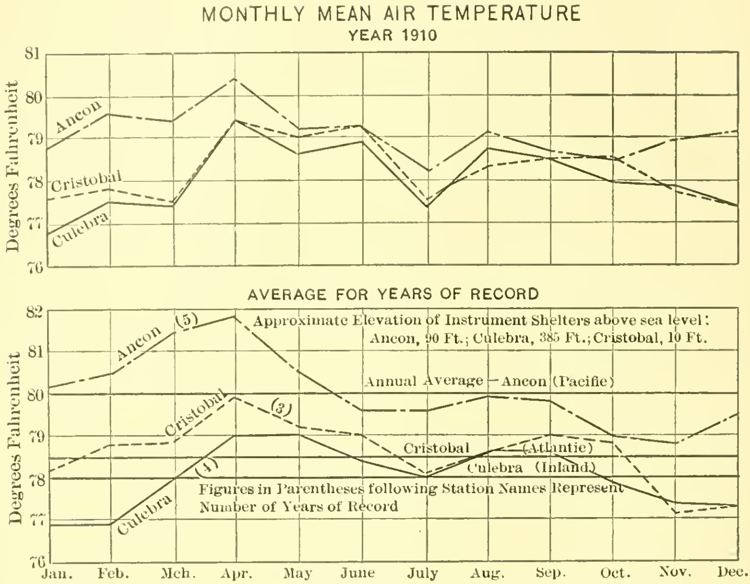

MONTHLY MEAN AIR TEMPERATURE
YEAR 1910
Degrees Fahrenheit
Ancon
Cristobal
Culebra
AVERAGE FOR YEARS OF RECORD
Approximate Elevation of Instrument Shelters above sea level:
Ancon, 90 Ft.; Culebra, 385 Ft.; Cristobal, 10 Ft.
Ancon (5)
Cristobal (3)
Culebra (4)
Annual Average—Ancon (Pacific)
Cristobal — (Atlantic)
Culebra (Inland)
Figures in Parentheses following Station Names Represent
Number of Years of Record
Jan. Feb. Mch. Apr. May June July Aug. Sep. Oct. Nov. Dec.

FIG. 4.

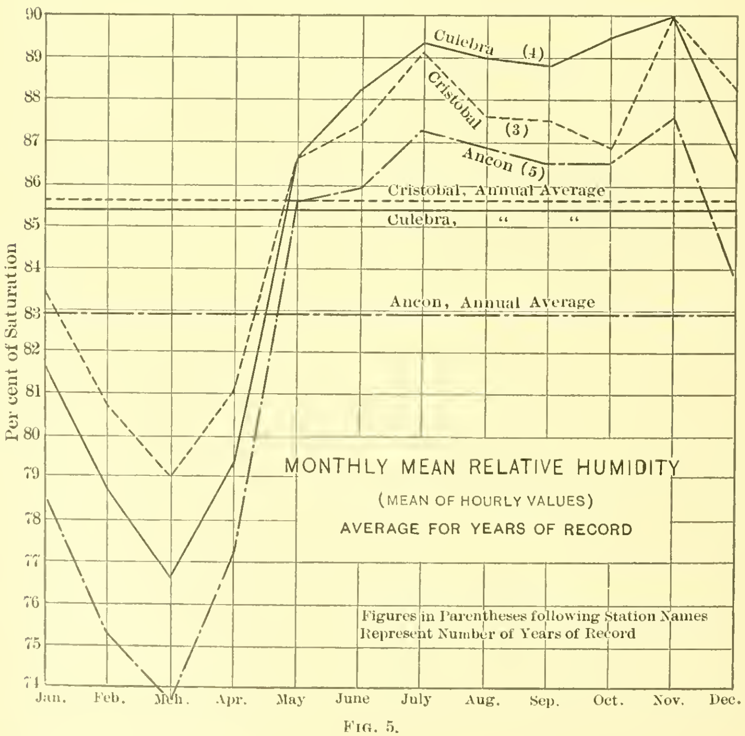

Per cent of Saturation
Culebra (4)
Cristobal (3)
Ancon (5)
Cristobal, Annual Average
Culebra, " "
Ancon, Annual Average
MONTHLY MEAN RELATIVE HUMIDITY
(MEAN OF HOURLY VALUES)
AVERAGE FOR YEARS OF RECORD
Figures in Parentheses following Station Names
Represent Number of Years of Record
Jan. Feb. Mch. Apr. May June July Aug. Sep. Oct. Nov. Dec.

FIG. 5.

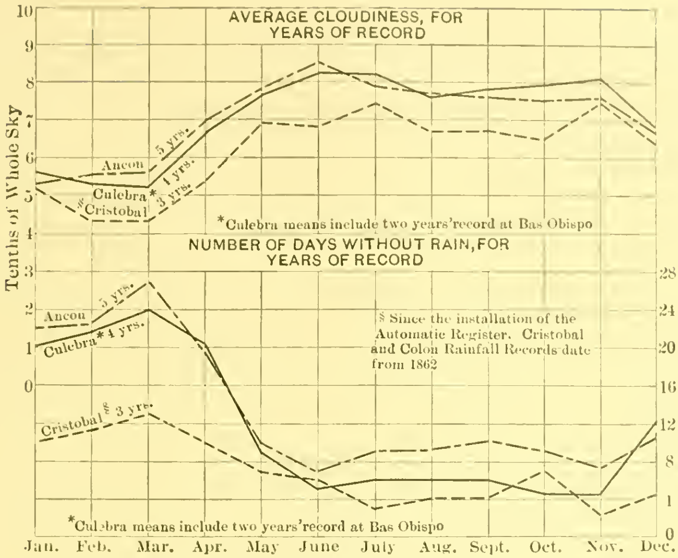

AVERAGE CLOUDINESS, FOR
YEARS OF RECORD
Tenths of Whole Sky
Ancon
5 yrs.
4 yrs.
3 yrs.
Culebra*
§Cristobal
*Culebra means include two years'record at Bas Obispo
NUMBER OF DAYS WITHOUT RAIN, FOR
YEARS OF RECORD
Ancon 5 yrs.
Culebra* 4 yrs.
§ Since the installation of the Automatic Register. Cristobal and Colon Rainfall Records date from 1862
Cristobal§ 3 yrs.
Days without Rain
*Culebra means include two years'record at Bas Obispo
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

Fig. 6.

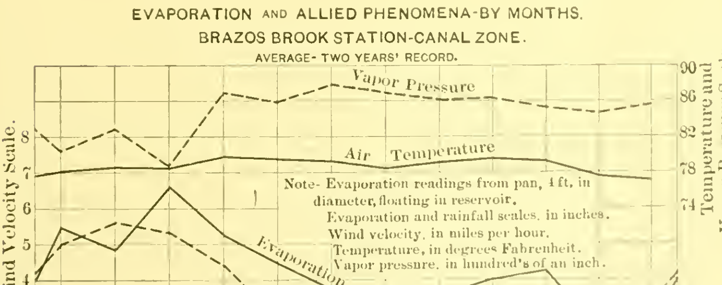

EVAPORATION AND ALLIED PHENOMENA-BY MONTHS.
BRAZOS BROOK STATION-CANAL ZONE.
AVERAGE- TWO YEARS' RECORD.
Vapor Pressure
Air Temperature
Note- Evaporation readings from pan, 4 ft. in diameter, floating in reservoir.
Evaporation and rainfall scales, in inches.
Wind velocity, in miles per hour.
Temperature, in degrees Fahrenheit.
Vapor pressure, in hundred's of an inch.
Evaporation
Average Hourly Wind Velocity
Rainfall
Evaporation and Wind Velocity Scale.
Temperature and Vapor Pressure Scale.
Rainfall Scale.
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

Fig. 7.

ing of the lake by about 1.7 ft. during the entire dry season, with no allowance for increase by rainfall on the surface. The high rates are recorded for short periods—⅕ in. and more for 40 days, and ¼ in. per day for about 10 days. The average evaporation for the 8 rainy months is about 0.11 in. per day, and during this period the rate is exceeded on only about 120 days.

Regarding the loss of water from the lake by trees growing in the shallow portions, it has been found by experience that all standing timber now growing on lands which are to be submerged will be killed

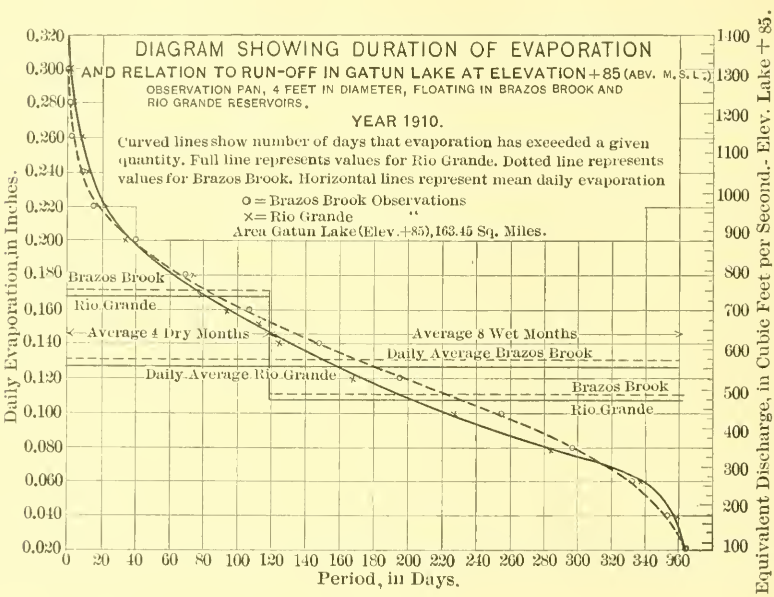
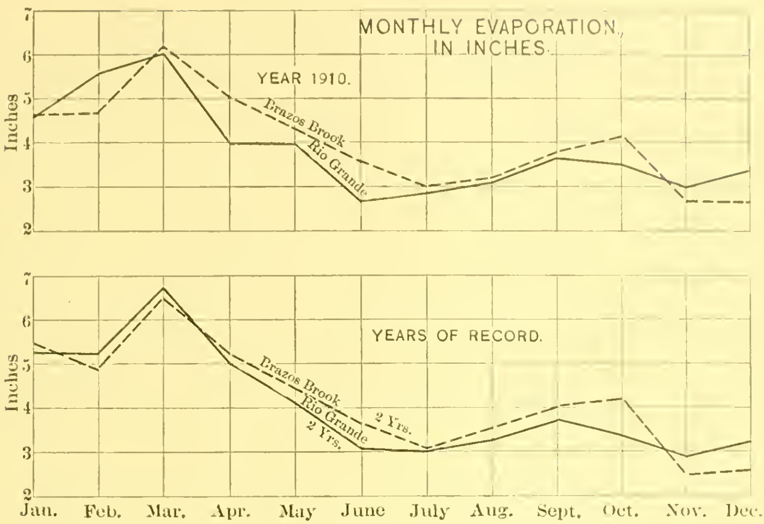


FIG. 8.

by the water after 2 or 3 years, and a large area, approximately that of the lake itself, will then be relieved of what must be a very large draft necessary for the support of this vegetation. Attention was first called to this gain to Gatun Lake by Lieut.-Col. H. F. Hodges, M. Am. Soc. C. E., Assistant Chief Engineer, Isthmian Canal Commission.*

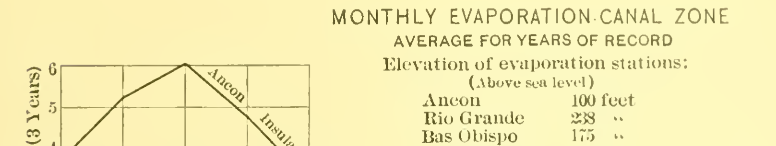
The diagram, Fig. 9, shows the monthly evaporation values for Brazos Brook and Rio Grande Reservoirs, and in Table 3 may be

* In his discussion of the paper entitled "Water Supply for the Lock Canal at Panama," by Julio F. Sorzano, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVII, p. 97.



MONTHLY EVAPORATION, IN INCHES.
YEAR 1910.
Brazos Brook
Rio Grande
YEARS OF RECORD.
Brazos Brook
2 Yrs.
Rio Grande
2 Yrs.
Inches
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 9.



MONTHLY EVAPORATION CANAL ZONE
AVERAGE FOR YEARS OF RECORD
Elevation of evaporation stations:
(Above sea level)
Ancon 100 feet
Rio Grande 238 "
Bas Obispo 175 "
Brazos Brook 48 "
Cristobal 9 "
Ancon Insulated Tank 10" in diameter
Rio Grande - Floating pan 4' in diameter
Bas Obispo - Concrete Tank 10" in diameter
Brazos-Brook - Floating pan 4' in diameter
Cristobal - Insulated Tank 10" in diameter
Evaporation, in Inches.
Ancon (3 Years)
Bas Obispo (2 Years)
Cristobal (3 Years)
Rio Grande (2 Years)
Brazos Brook (2 Years)
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct Nov. Dec.

FIG. 10.

TABLE 3.—EVAPORATION IN

	1907.		1908.							
	‡BAS OBISPO.		†ANCON.		‡BAS OBISPO.		†CRISTOBAL.		†ANCON.	
	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.
January	5.175	0.167	3.348	0.108	5.617	0.181	4.868	0.157	3.348	0.108
February	5.072	0.181	6.557	0.226	5.729	0.198	7.958	0.274	3.728	0.133
March	6.538	0.211	6.997	0.226	6.290	0.203	7.557	0.244	4.931	0.159
April	6.486	0.216	5.921	0.197	5.475	0.182	6.930	0.231	3.883	0.129
May	4.681	0.151	3.219	0.104	3.175	0.102	2.570	0.083	2.451	0.079
June	3.125	0.104	3.046	0.101	3.415	0.114	3.184	0.106	1.916	0.064
July	3.152	0.102	3.017	0.097	3.250	0.105	2.822	0.091	2.080	0.067
August	3.582	0.116	3.203	0.103	3.425	0.110	3.192	0.103	2.271	0.073
September	3.358	0.112	3.049	0.102	3.635	0.121	3.327	0.111	2.484	0.083
October	2.938	0.095	3.256	0.105	3.875	0.125	3.324	0.107	2.826	0.091
November	3.599	0.120	2.414	0.080	2.730	0.091	2.700	0.090	2.332	0.078
December	4.896	0.158	2.942	0.095	3.445	0.111	4.056	0.131	3.020	0.097
Annual	52.602	......	46.996	......	50.061	......	52.488	......	35.270	......
Sums, Apr. to Dec.	35.817	......	30.067	......	32.425	......	32.105	......	23.263	......
Mean, 12 months	4.384	0.144	3.914	0.129	4.172	0.137	4.374	0.144	2.939	0.097
Mean, 9 months	3.980	0.130	3.341	0.109	3.603	0.118	3.567	0.117	2.585	0.085

* Tank floating in lake; 4 ft. diameter. † Insulated tank, protected from direct rays of

found the monthly values for all stations. Evaporation measurements were begun on the Isthmus in 1907, and have been continued since. The best values are being obtained from copper pans, 4 ft. in diameter, floating in Rio Grande and Brazos Brook Reservoirs and in Gatun Lake. At these stations the evidence is being made to approximate as nearly as possible the conditions of an extensive water surface fairly exposed to all atmospheric conditions. Fig. 1, Plate I (from Annual Report, I. C. C., 1910-11), shows the evaporation apparatus at Rio Grande Reservoir, which is similar to those in use at Brazos Brook and Gatun.

In order to obtain some data regarding the relative quantities evaporated by day and by night, series of readings were taken at different stations, and the results appear in Table 5. It has seemed desirable to obtain some information as to the relative evaporation that will take place in various parts of the lake due to the differing conditions which will exist along the shore. In order to investigate this matter, three 4-ft. floating pans were placed in Gatun Lake in such locations as seemed to cover the various conditions to be anticipated. Pan No. 1 was placed in an exposed location about 1 000 ft. south of

THE CANAL ZONE, IN INCHES.

1909.								1910.					
*Rio Grande.		*Brazos Brook.		†Cristobal.		†Ancon.		*Rio Grande.		*Brazos Brook.		*Cristobal.	
Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.	Total.	Daily mean.
......	......	......	......	4.868	0.157	4.351	0.140	4.612	0.149	4.622	0.149	5.257	0.170
......	......	......	......	5.353	0.191	5.350	0.191	5.529	0.197	4.668	0.167	4.574	0.163
......	......	......	......	6.445	0.208	6.190	0.200	6.063	0.194	6.151	0.198	6.500	0.210
5.960	0.199	5.366	0.179	6.468	0.216	4.379	0.146	3.986	0.133	5.025	0.168	5.209	0.174
4.205	0.136	4.597	0.148	3.868	0.125	3.378	0.109	3.916	0.126	4.304	0.139	3.493	0.113
3.417	0.114	3.806	0.127	2.498	0.083	2.666	0.089	2.654	0.088	3.516	0.117	2.703	0.090
3.117	0.101	3.042	0.098	2.445	0.079	2.740	0.088	2.846	0.092	3.014	0.097	2.359	0.076
3.353	0.108	3.760	0.121	2.636	0.085	2.866	0.092	3.096	0.100	3.189	0.103	2.852	0.092
3.768	0.126	4.169	0.139	2.727	0.091	3.078	0.103	3.677	0.123	3.804	0.127	2.791	0.093
3.094	0.100	4.168	0.134	3.027	0.098	3.436	0.111	3.577	0.115	4.177	0.135	3.343	0.108
2.713	0.090	2.152	0.072	1.950	0.065	3.132	0.104	2.999	0.100	2.718	0.091	2.817	0.091
2.993	0.096	2.379	0.077	2.746	0.089	3.528	0.114	3.442	0.111	2.746	0.089	3.732	0.120
......	......	......	......	45.031	......	45.097	......	46.337	......	47.934	......	45.630	......
32.620	......	33.439	......	28.365	......	29.203	......	30.193	......	32.493	......	29.299	......
......	......	......	......	3.753	0.124	3.758	0.124	3.861	0.127	3.994	0.131	3.802	0.125
3.624	0.119	3.715	0.122	3.152	0.103	3.245	0.106	3.355	0.110	3.610	0.118	3.255	0.107

sun; 10 in. diameter. ‡ Concrete tank, 12½ ft. diameter, 5 ft. deep.

the dam and an equal distance from each shore or other obstruction. This exposure was intended to represent ordinary conditions on the open lake. Pan No. 2 was placed among the trees of what had been a considerable forest, but which was now partly submerged by the rising waters of the lake. It is located about 500 ft. in from the edge of the forest, several trees being felled in order that the records might not be affected by rain dropping from the leaves. Pan No. 3 was placed in the center of an area of tall water grass, about ½ acre in extent. Such a growth as this will be very frequent along the shallow portions of the shore. In the last two exposures it appears that the action of the wind is a very great factor in determining the quantity of evaporation. It also demonstrates that action of the shore growth will be that of a wind-break, and that the evaporation figures based on data collected from exposed locations are on the safe side, that is, probably they will give too great an evaporation. Table 4 shows the results obtained since the installation of the pans.

The equipments at Rio Grande, Brazos Brook, and Gatun consist of the following measuring instruments:

A copper pan, 4 ft. in diameter and 10 in. deep, floating in the

water, for evaporation; an anemometer, for wind movement directly over the evaporation pan; a rain gauge, for rainfall; a thermometer, for water temperatures, and maximum and minimum thermometers, exposed in an instrument shelter of the regular type located on the shore close by, for recording the air temperatures.

The evaporation pan is set in a light wooden frame, properly buoyed, which serves to keep the top of the pan above the surface of the water in the lake, as well as to prevent occasional waves from splashing over into the pan. The pan rests with its bottom and sides submerged in the water, thus insuring that the temperature of the water within the pan at all times will be very close to that of the water at the surface. The float is located at a considerable distance from shore, for the purpose of getting an exposure which represents actual average conditions.

A sharp-pointed, copper-wire index is set in the center of the pan and inside a perforated copper cylinder still-box which serves to reduce the wave motion of the surrounding water, and thus permits more accurate readings of the daily evaporation. The index point is 4 in. below the top of the pan, and it follows that the usual height of the water in the pan is also approximately 4 in. below the top. This arrangement is necessary to prevent the overflow of the pan during heavy rains.

TABLE 4.—Evaporation Records for Pans Floating in Gatun Lake.

Monthly Values.

Month.	Pan No. 1.			Pan No. 2.			Pan No. 3.		
	Rainfall, in inches.	Wind average, in miles per hour.	Evaporation, in inches.	Rainfall, in inches.	Wind average, in miles per hour.	Evaporation, in inches.	Rainfall, in inches.	Wind average, in miles per hour.	Evaporation, in inches.
1911.									
May*	15.33	3.93	2.12	14.21	1.47	1.75	14.41	....	1.83
June	16.48	3.86	2.80	17.65	0.97	2.41	17.89	....	2.48
July	7.20	5.90	4.56	7.31	1.47	3.28	7.33	....	2.78
August	8.19	4.74	4.11	6.77	0.86	3.07	7.35	....	3.37
September	4.83	5.02	4.58	7.10	1.04	2.63	6.96	....	3.13
October	16.85	4.11	3.53	16.62	0.68	2.77	15.21	....	3.00

* No record for the first 10 days of May.

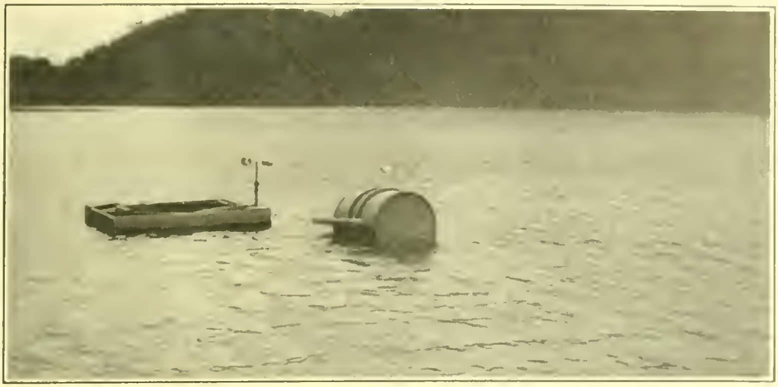


FIG. 1.—EVAPORATION GAUGE IN LAKE.

FIG. 2.—CABLE ANCHORAGE AND GAUGING CAR, ALHAJUELA.

FIG. 3.—FLUVIOGRAPH, ALHAJUELA STATION.

TABLE 5.—EVAPORATION IN CANAL ZONE. COMPARATIVE VALUES FOR DAY AND NIGHT.

Month.	1908.*						1909.				1911.*	
	Ancon.		Bas Obispo.		Cristobal.		Ancon.		Cristobal.		Cristobal.	
	Day.	Night.	Day.	Night.	Day.	Night.	Day.	Night.	Day.	Night.	Day.	Night.
January	......	......	......	......	......	.....	1.755	1.593	2.754	2.114	4.600	3.565
February	5.062	1.495	3.605	2.124	4.200	3.758	2.134	1.594	2.916	2.437	3.763	3.341
March	5.566	1.431	3.919	2.371	4.402	3.155	2.693	2.238	3.660	2.785	4.810	4.275
April	4.404	1.517	3.630	1.845	3.999	2.931	2.108	1.775	3.594	2.873	3.152	2.247
May	2.001	1.218	1.715	1.460	1.706	0.864	1.317	1.131	2.268	1.600	1.973	1.323
June	2.034	1.012	1.955	1.460	1.878	1.306	0.941	0.975	1.332	1.166	1.674	1.276
July	1.900	1.117	1.670	1.580	1.584	1.238	1.078	1.002	1.252	1.193	......	......
August	2.175	1.028	1.702	1.723	1.985	1.207	1.241	1.030	1.467	1.169	......	......
September	1.924	1.125	2.178	1.457	2.096	1.231	1.425	1.059	1.706	1.021	.....	......
October	2.051	1.205	2.467	1.408	2.016	1.308	1.862	0.964	1.784	1.243	......	......
November	1.362	1.052	1.555	1.175	1.572	1.128	1.395	0.937	1.129	0.821	......	......
December	1.337	1.605	2.040	1.405	2.252	1.804	1.775	1.245	1.484	1.262	......	......
Total	29.816	13.805	26.436	18.008	27.690	19.930	19.724	15.546	25.346	19.684	19.972	16.027
Percentage	68	32	60	40	58	42	56	44	56	44	55	45

* 11 months, 1908; Cristobal, 6 months, 1911.
Readings taken at 8.00 A. M. and 8.00 P. M. daily.
Mean of all observations. 59% day and 41% night.

The pan is set by bringing the surface of the water exactly to the level of the point of the copper index. When the evaporation measurements are made 24 hours later, if there has been no rainfall, the surface of the water in the pan will be found below the index point. The quantity of water necessary to raise the water surface to the level of the index point is carefully measured and poured into the pan. This quantity of water, expressed in thousandths of an inch over the surface of the evaporation pan, equals the evaporation for the previous 24 hours. When the rainfall during the day has exceeded the evaporation, the surface of the water in the pan will be found above the point of the copper index. The excess water must be removed from the pan and measured. In this case the difference between the rainfall and the quantity of water removed from the pan, expressed in thousandths of an inch, equals the evaporation for the previous 24 hours. When rain occurs during the day, but is less than the evaporation, the sum of the rainfall and the quantity of water poured into the pan, expressed in thousandths of an inch, equals the evaporation. The water poured into or removed from the evaporation pan is measured in a copper measuring tube having exactly $\frac{1}{100}$ of the cross-section

area of the evaporation pan. Depths are measured with the regular rainfall measuring stick, graduated to inches and tenths; $\frac{1}{10}$ in. in the measuring tube represents $\frac{1}{1000}$ in. over the surface of the evaporation pan.

Table 5 shows the quantities of evaporation, by day and by night, that have been recorded at the several stations on the Canal Zone, together with the relative percentages. The mean of the entire series is 59% by day and 41% by night.

Besides the direct loss due to the run-off by evaporation from water surfaces, a large percentage of the rainfall is still unaccounted for, which is aptly called "retention" by Professor D. W. Mead,* M. Am. Soc. C. E.; part of this is used by vegetation and transpired by the leaves, part is retained in the ground and returned to the stream at periods of low rainfall, and a portion is permanently lost to the watershed either by deep percolation or, as is sometimes the case, by meeting a permeable stratum which conducts it to another basin. This latter cause, however, is usually relatively unimportant, but should not be overlooked in the examination of a water supply. In the Gatun Lake Basin, due to the practically impervious argillaceous sandstone and other rocks of similar character which appear to underlie the entire lake area, permanent loss of water to the basin is probably too small for any consideration, and is neglected hereafter in this paper. The following shows the portion of the rainfall unaccounted for in certain northern streams:

Rivers.	Portion of rainfall unaccounted for.
Hudson, N. Y.	52 per cent.
Sudbury, Mass	49 " "
Croton, N. Y.	46 " "
Wisconsin, Wis.	65 " "
Chippewa, "	50 " "
Rock, "	21 " "

In his treatise on water supply engineering, the late J. T. Fanning, M. Am. Soc. C. E., has given values from investigations at Emdrup, Denmark, from which the following is deduced:

* "Flow of Streams and the Factors that Modify It," *Bulletin*, Univ. of Wisconsin, No. 425.

	Mean evaporation.	Ratio to water evaporation.
Water surfaces, mean evaporation (annual)	27.9 in.	1.00
Short grass	30.1 "	1.08
Long grass	44.0 "	1.58
From Bradmore's "Hydrology."		
Lancashire, England, Lat. 53° 30′ N. from earth	25.7 "	....
Cumberland, England, Lat. 54° 34′ N., from earth	29.2 "	....

Investigations by Risler concerning the quantity of water required for different crops* are tabulated below, and give the daily consumption of water for various crops:

Lucerne grass	from	0.134	to	0.267	in.
Meadow "	"	0.122	"	0.287	"
Oats	"	0.140	"	0.193	"
Indian corn	"	0.110	"	1.570	"
Clover	"	0.140	"	....	"
Vineyard	"	0.035	"	0.031	"
Wheat	"	0.106	"	0.110	"
Rye	"	0.091	"	....	"
Potatoes	"	0.038	"	0.055	"
Oak trees	"	0.038	"	0.030	"
Fir trees	"	0.020	"	0.043	"

Mr. Tweeddale states as his opinion that cereals and grasses, from seed time to harvest, will take up, respectively, from 15 to 37 in.

In his report on water supply, Mr. C. C. Vermeule has used the following factors for the water required by various crops in one growing month:

Crop.	Inches.
Forest (oak and chestnut)	1.2
Wheat, rye, oats, etc	3.5
Indian corn	4.5
Potatoes, etc.	1.2
Long grass	6.0

* Quoted by W. Tweeddale, Kansas State Board of Agriculture, December 31st, 1889.

Crop.	Inches.
Short grass	5.0
Orchards	3.0
Fallow lands, etc	4.0

In investigation work at Pomona, Cal.,* the following duty has been required of the water used on various crops for the average year (1906-09):

Citrus fruits (oranges and lemons)	2.94 ft.
Deciduous fruits and miscellaneous crops	2.58 "
Alfalfa	4.6 "

While these figures do not show the quantity of water that similar crops would use were they entirely dependent on natural supply, they do indicate the relative quantities that such crops demand for the best conditions, and the quantity they might use if it were available. The depth of water used on a 21½-acre apple orchard in Wenatchee, Wash.,† was found to be 23 in. during the year, the trees being 7 years old and producing heavily. Mr. Samuel Fortier‡ states that the results of experiments have shown that, where water is applied to the surface of orchard soils, the evaporation is very great as long as the top layer remains moist. Even in light irrigation, the loss in 48 hours after the water is put on may amount to from 10 to 20% of the volume applied. Mr. W. W. McLaughlin,§ in discussing the irrigation of grain, states that it is always safe to assume that the ranker the growth of straw the greater will be the quantity of water required. He also says that the total depth of water applied to grain crops on the older lands will vary from 1 to 8 ft., depending on the porosity of the soil and the climate of the place, average depths of 2 and 3 ft. representing the quantities required, respectively, in cool and warm climates. Field peas‖ in an average irrigation season of 42 days are shown to have required about 4.7 in., or about 0.112 in. per day.

The foregoing figures, while not exact, and applying to crops and conditions elsewhere and under different climatic conditions, are nevertheless instructive in obtaining some idea of the quantity of water which must be required by the rank and lush vegetation of this

* *Bulletin 236*, Office of Exper. Station, U. S. Dept. of Agriculture, p. 87.

† *Farmers' Bulletin 404*, U. S. Dept. of Agriculture, p. 26.

‡ *Farmers' Bulletin 404*, U. S. Dept. of Agriculture, p. 28.

§ *Farmers' Bulletin 399*, U. S. Dept. of Agriculture, p. 18.

‖ *Bulletin 72*, Wyoming Experiment Station, p. 7.

tropical country. With all its heavy precipitation, were it not for the steep hillsides, and consequent quick discharge of the waters, it is doubtful if the ratio of run-off to rainfall might not even be less here than in countries where the rainfall is smaller but the vegetation less luxuriant.

E. Oppokov* states that marshes are a detriment to the drainage area of a water supply system. It is ordinarily thought that such areas are valuable for the storage of water in the rainy season, and that during the dry period they allow of a gradual discharge, which increases the stream flow. Having studied the features of the Dnieper and adjoining rivers, Mr. Oppokov believes that his investigations prove there is intensified evaporation from marsh areas in periods of drought, which not infrequently results in the total drying out of the marsh. These areas then act as a great sponge for the absorption of subsequent precipitation and seepage waters from the uplands, and cause decreased returns of the waters to the rivers, the run-off being actually lessened thereby.

In the case of the Gatun Lake water-shed, there will probably be an increase in run-off when the swamp areas are eliminated by submergence. Under present conditions, these areas annually absorb large quantities of the rainfall to make up for the depletion caused by evaporation and plant necessity. After submergence, the trees and other vegetation will die off, and the ground will become saturated once for all. Subsequent rainfall will then be available for lake purposes. Such conditions will be especially appreciated during periods of light rainfall, when the demands of vegetation are greatest.

The fact that a considerable rainfall may occur with little run-off from a basin overgrown with vegetation, and that the daily evaporation from a water surface is relatively small in comparison with the precipitation, is an indication that the available run-off will be increased. For river regulation and the prevention of floods, this condition might present an entirely different aspect than when applied to the conservation of the water. The well-sustained run-off from the area above Alhajuela is due, not only to its large rainfall, but probably also to the absence of swamps with their shallow storage, and the presence of an extensive limestone formation. This latter undoubtedly allows the deep storage of large quantities of water during the rainy season,

* *Russ. Jour. Expr.*, Landau, 1910, pp. 369-372.

which, as the dry season proceeds, is given off for the benefit of river discharge. The swamp and the limestone storage, although both classed as underground, to distinguish them from surface water, are nevertheless entirely distinct in their tendencies. The former is shallow and readily affected by the agencies producing evaporation, including plant demands; the latter, deep and far removed from the surface losses, is all available for stream flow, if it can but find an outlet.

An excellent example of this latter condition is found in a limestone cliff overhanging the Chagres River above Alhajuela. Here is a nearly perpendicular wall, more than 100 ft. above the river surface, and having a length of more than 500 ft. This whole surface, more or less covered with moss and clinging vegetation, is constantly and copiously dripping with water during the entire year. The contour surveys and explorations have shown that there are no extensive swamp areas in the Chagres River Basin above Elevation + 60, and it seems fair to presume that, when the lake has reached its final elevation, hydrographic conditions similar to those now observed for the area above Alhajuela will obtain elsewhere, modified by local conditions of rainfall.

The geological and topographical investigations which have been made in the lake area and its drainage basin seem to give assurance of exceedingly favorable conditions for the collection of the water supply and its conservation. The lower part of the Chagres Basin, as has been stated, is underlaid by argillaceous sandstone, and this is the principal constituent in the ridges bordering the lake. Nearly the entire bottom of the lake, up to about Elevation + 30, is covered with a deposit of alluvium, which is also practically impervious. This cover, about 51 miles in extent, and of varying thicknesses up to several hundred feet, presents an additional safeguard against bottom seepage, and at the place where, if needed at all, it will be of most service, namely, in the deepest parts of the lake. Above this impervious covering is found the residual deposit overlying the impervious argillaceous sandstone from which it was derived. The characteristics of this surface layer are its more or less claylike nature at the extreme surface, and its sandy and less cohesive structure as it approaches the solid rock. The nature of this argillaceous sandstone is to crumble and break down to a fine clay where mostly exposed to the weather, the

particles becoming larger and more firm with distance from the surface. The result of this quality is that there is an extensive layer of more or less open structure overlying the impervious rock, capable of storing large quantities of water, and well protected from surface evaporation. From this layer comes the well-sustained flow that appears in many streams during periods when there is little or no rain. This formation also gives additional storage capacity to the lake, so that demands on it will be honored in excess of visible storage. Such condition has been found in the case of the Rio Grande Reservoir, and will be commented on later. What this storage may be is unknown at present, and can only be determined after the lake is filled. This quantity should not be considered in calculations affecting the water supply, but should be left as an asset to be applied to any losses which may have been overlooked or under-estimated.

Rainfall.

The cause of rainfall is to be found in the operation of certain general principles of physics, but is influenced more or less by local conditions. Rainfall is due to the precipitation of the water vapor in the atmosphere which has been derived primarily from the ocean by the process of evaporation. No matter how far this vapor may be driven by the winds, in the course of time the water returns again to the ocean. Evaporation takes place from the surface of every body of water, from the ground, and from vegetation. Whenever for any cause the moisture-laden air is cooled, condensation and consequent precipitation take place.

According to Curtis, three different processes are primarily concerned in the production of rain, acting either singly or in combination. These factors are:

1.—Convective currents;

2.—Hills and mountains which cause deflection of atmospheric currents; and

3.—Cyclonic circulation.

The Isthmus of Panama is in that portion of the globe where the influence of convection is very great, but where cyclonic disturbances are almost absent. Currents due to convection of heat are nearly vertical, and moisture evaporated in a region exposed to these con-

ditions will largely be precipitated before being carried far away. Increased evaporation, therefore, will be followed by increased rainfall, and any change that increases or decreases the former will be followed by a corresponding change in the precipitation. Wherever there are mountain ridges the slopes of which obstruct and deflect upward the prevailing winds, there also will be found a well-watered country and perennial streams. So effective is this cause in producing rainfall that it is of frequent occurrence to find luxuriant vegetation on the windward side of a mountain range while the leeward side is dry and parched, and with meager growth. Although not to extremes, such conditions as these are found in the Canal Zone, with its 72 in. of

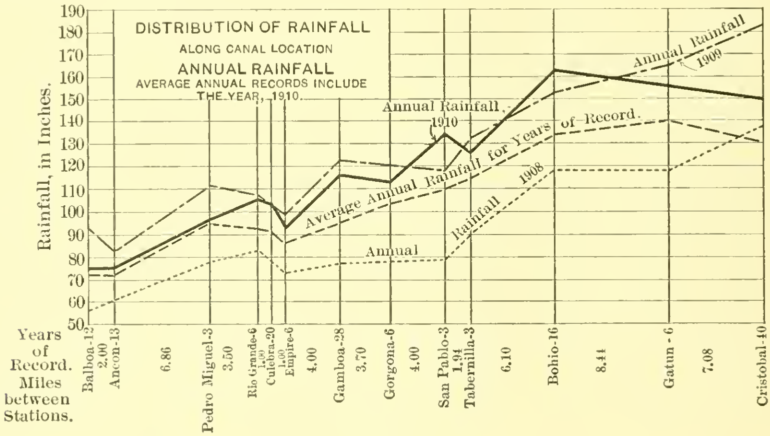


Fig. 11.

precipitation on the Pacific Coast and its 130 in. on the Atlantic, which are averages of long-period records.

The conditions of rainfall following excessive evaporation in this region are best shown in the basin above Alhajuela. There, on the high mountains, the moisture-laden winds from the north and west are forced to drop their loads. Along the Canal Zone and to the southwest, the ridges are of much less elevation, and therefore do not so much disturb the direction of the wind. The results of these conditions are found in the run-off at Alhajuela, with its well-sustained flow, and at Trinidad and Caño, with the opposite conditions.

The distribution of rainfall across the Isthmus is shown by Fig. 11. Culebra, with its annual average of 91 in., is at the limit of the wind-

ward slope, and its rainfall, averaged with that of Cristobal and Gatun, gives a rough approximation of the rainfall conditions north of the divide and over the area in which will lie Gatun Lake and its drainage area. On the southerly or leeward side of the ridge the rainfall averages about 80 in. annually, or less than three-fourths of that on the windward side. Due to other local conditions, however, the actual rainfall on the Gatun Basin is larger than these mean values and more nearly approaches a ratio of 3 to 2.

Records of rainfall have been kept on the Isthmus of Panama in the vicinity of the Canal Zone for long periods at several of the stations. Table 6 gives the stations which have been in operation for 6 years or more, the length of record, and the probable accuracy of the record as a basis for prediction of the average conditions which may be expected in the future.

TABLE 6.—Rainfall Stations in Canal Zone.

Station.	Length of record, in years.	Probable error. Percentage.	Station.	Length of record, in years.	Probable error. Percentage.
Cristobal.........	40	1.75	Panama (Ancon).	13	7.00
Gamboa.........	28	2.25	Empire...........	6	14.00
Culebra..........	20	3.25	Gatun....	6	14.00
Bohio............	16	4.50	Gorgona..........	6	14.00
Alhajuela	13	7.00	Rio Grande.......	6	14.00

The figures in Table 6 for probable deviation from the actual average agree substantially with those given by Mr. A. R. Binnie, and quoted by Mr. Rafter.*

In his report on the "Influence of Forests on Stream Flow," Lieut.-Col. Edward Burr, M. Am. Soc. C. E., Corps of Engineers, U. S. A., states as his deduction that 60 years or more of climatic records are necessary from which to draw satisfactory conclusions, and that those drawn from records of 40 years or less will probably be misleading or incorrect. Col. Burr's conclusions regarding length of records necessary for basing estimates are therefore substantially in accord with the similar deductions of Messrs. Binnie and Henry, mentioned previously. From an examination of the curve, Fig. 12, it appears, however, that the probable variation from normal, in records of from 35 to 40 years, is of such small amount that errors due to observation and

* Water Supply and Irrigation Paper No. 85, p. 18.

TABLE 7.—RAINFALL ON THE

Average Quantities for

Station.	Station Average.	January.		February.		March.		April.		May.	
	No. of years.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.
Ancon	13	1.12	1.12	0.76	1.88	0.89	2.77	2.47	5.24	8.66	13.90
Balboa	12	1.28	1.28	0.36	1.64	0.99	2.63	3.92	6.55	6.94	13.49
Miraflores	2	3.88	3.88	1.92	5.80	1.80	7.60	3.72	11.32	9.57	20.89
Pedro Miguel	3	1.64	1.64	1.25	2.89	0.72	3.61	2.88	6.49	12.16	18.65
Rio Grande	6	1.79	1.79	0.69	2.48	0.42	2.90	3.05	5.95	10.54	16.49
Culebra	20	1.92	1.92	0.56	2.48	0.75	3.23	3.78	7.01	11.11	18.12
Camacho	4	1.77	1.77	0.89	2.66	0.82	3.48	2.92	6.40	11.00	17.40
Empire	6	1.02	1.02	0.59	1.61	0.57	2.18	3.65	5.83	9.52	15.35
Gamboa	28	1.98	1.98	0.85	2.83	0.88	3.71	3.62	7.33	10.89	18.22
Alhajuela	11	1.44	1.44	0.69	2.13	0.78	2.91	3.34	6.25	12.35	18.60
El Vigia	2	2.30	2.30	3.64	5.94	1.64	7.58	2.32	9.90	13.00	22.90
Gorgona	6	2.37	2.37	0.76	3.13	1.30	4.43	3.18	7.61	12.20	19.81
San Pablo	3	2.37	2.37	1.57	3.94	1.81	5.75	5.14	10.89	10.09	20.98
Tabernilla	3	2.59	2.59	1.80	4.39	2.39	6.78	4.99	11.77	11.34	23.11
Bohio	16	6.38	6.38	2.12	8.51	2.08	10.59	5.84	16 43	13.93	30.36
Trinidad	3	4.86	4.86	3.70	8.56	4.41	12.97	6.55	19.52	13.47	32.99
Monte Lirio	3	5.01	5.01	5.33	10.34	5.71	16.05	5.94	21.99	14.56	36.55
Gatun	6	4.75	4.75	2.44	7.19	3.49	10.68	3.64	14.32	15.39	29.71
Brazos Brook	4	4.87	4.87	2.38	7.25	4.40	11.65	3.56	15.21	11.92	27.13
Cristobal	40	4.13	4.13	1.45	5.58	1.70	7.28	4.19	11.47	12.31	23.78
Porto Bello	3	12.57	12.57	4.74	17.31	3.60	20.91	8.56	29.47	13.34	42.81
Nombre de Dios	2	......	......	......	......	......	......	......	......	......	......

the application of the data from a few stations to a large area may somewhat offset the accuracy presumed for longer records.

Fig. 13 shows graphically the annual rainfall for five stations having the longest records, and Fig. 16 shows the comparative monthly distribution for several stations, including those recorded on Fig. 13. Table 7 gives the mean monthly and accumulated quantities for each station.

Of the 31 rainfall stations now operated, 17 are supplied with rain gauges of the U. S. Weather Bureau standard pattern, 11 have automatic gauges of the tipping-bucket type, and 3 have 14-day automatic gauges of Dutch make. The latter are located in isolated positions, where daily attendance is impracticable.

The Weather Bureau standard rain gauge consists of a hollow metal cylinder, 8 in. in diameter and 20 in. high, made in two sections, a receiver for catching the rainfall and an 8-in. overflow tank for storing any water that may accumulate in excess of the quantity required to fill the measuring tube. The receiver leads by a funnel to the copper measuring tube, approximately 2½ in. in diameter, placed inside the

ISTHMUS OF PANAMA, IN INCHES.
the Years of Record.

June.		July.		August.		September.		October.		November.		December.	
Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.	Current month.	Accumulated.
8.69	22.59	8.18	30.77	7.64	38.41	7.49	45.90	10.43	56.33	10.94	67.27	4.40	71.67
8.36	21.85	10.11	31.96	7.54	39.50	6.36	45.86	9.12	54.98	9.63	64.61	6.45	71.06
16.05	36.91	10.79	47.73	9.22	56.95	10.70	67.55	13.26	80.91	13.34	94.25	10.88	105.13
12.11	30.76	9.22	39.98	9.82	49.80	8.49	58.29	13.10	71.39	13.02	84.41	10.39	94.80
10.68	27.17	12.03	39.20	10.22	49 42	11.29	60.71	13.11	73.82	11.39	85.21	7.02	92.23
9.17	27.29	9.68	36.97	10.57	47.54	11.35	58.89	11.35	70.21	12.52	82.76	8.19	90.95
12.07	29.47	11.93	41.40	10.25	51.65	11.33	62.98	13.71	76.69	14.70	91.39	8.34	99.73
8.57	23.92	10.24	34.16	10.14	44.30	7.93	52.23	14.16	66.39	11.49	77.88	6.61	84.49
9.72	27.94	10.42	38.36	12.20	50.56	10.61	61.17	12.77	73.94	12.54	86.48	7.31	93.79
13.13	31.73	13.85	45.58	13.29	58.87	12.00	78.87	13.53	84.40	14.70	99.10	8.04	107.14
14.48	37.38	14.63	52.01	13.12	65.13	15.39	80.52	17.21	97.73	20.64	118.37	10.20	128.57
9.02	28.83	11.99	40.82	12.37	53.19	13.17	66.36	13.10	79.46	14.37	92.83	8.41	102.24
10.69	31.67	12.21	43.86	10.94	54.80	12.72	67.62	16.14	83.66	15.14	98.80	9.88	108.68
10.57	33.68	10.43	44.11	10.80	54.91	14.41	69.32	18.19	87.51	16.38	103.89	9.85	113.74
12.46	42.82	13.37	56.19	15.34	71.53	14.02	85.55	16.80	102.35	19.39	121.74	11.25	132.99
11.61	44.60	10.72	55.32	13.51	68.83	14.21	83.04	14.49	97.53	23.47	121.00	15.79	136.79
14.38	50.93	16.16	67.09	12.65	79.74	13.32	93.06	14.46	107.52	28.63	136.15	14.74	150.89
13.38	43.09	13.68	56.77	16.10	72.87	10.91	83.78	16.91	100.69	24.03	124.72	15.83	140.55
15.03	42.16	17.45	59.61	15.24	74.85	12.31	87.16	14.76	101.92	26.43	128.35	17.56	145.91
13.23	37.01	16.50	53.51	15.22	68.73	12.56	81.29	14.15	95.44	22.01	117.45	12.58	130.03
16.65	59.46	19.70	79.16	17.17	96.33	11.91	108.24	8.51	116.75	31.85	148.60	30.07	178.67
......	......	17.02	......	10.89	......	7.31	......	11.89	......	20.30	......	22.43	......

8-in. cylinder. When the measuring tube has become filled, the water is allowed to overflow into the larger cylinder, whence it is drawn off later and measured. The two cylinders are proportioned so that the measuring tube has exactly one-tenth of the cross-section area of the receiving cylinder, therefore $\frac{1}{10}$ in. in depth in the measuring tube represents 0.01 in. of rainfall. The quantity of rainfall is obtained by measuring the depth in the smaller cylinder with a graduated stick, turning back into the smaller cylinder any rain which may have overflowed into the larger one.

The tipping-bucket gauge is of more complicated design. It is composed of a metal cylinder, 12 in. in diameter, open at the top, and about 30 in. high over all. Inside the cylinder, a brass bucket is balanced, having two compartments of the same size and shape. The bucket is adjusted to tip when a quantity of water equal to $\frac{1}{100}$ in. over the area of the receiving cylinder has accumulated, and spill the contents into the cylinder, where it can be measured with a stick as a check. The bucket is connected electrically with a register on a revolving drum on which is wrapped a sheet of paper graduated in

hour- and five-minute periods. Every tip of the bucket is registered on the paper by a pen or pencil, and as each mark represents $\frac{1}{100}$ in. of rainfall, an automatic record is obtained of the time the storm begins and ends, while the number of marks gives the total quantity and the rate of downpour in a given time.

The Dutch gauges are of somewhat similar construction, except that there is no electrical connection, the entire gauge and recording devices being contained in the same case. On account of direct measurements, the standard 8-in. rain gauge is considered the most reliable indicator of the quantity of precipitation. In order to obtain

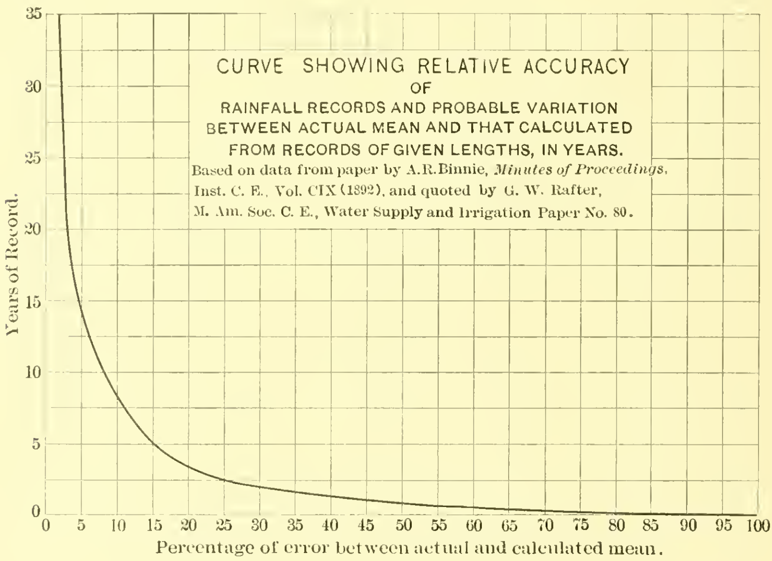


FIG. 12.

some idea of the reliability of the automatic gauge, a standard gauge (8-in. diameter) and an automatic gauge (12-in. diameter), were located side by side and daily readings were taken during August and September, 1910. During August, 95% of the measurements in the two gauges agreed exactly, and the remaining 5% were only slightly at variance with each other. The total monthly rainfall by stick measurement was the same in both gauges. During September, 93% of the measurements agreed exactly, while the total monthly rainfall by the standard gauge equalled 101% of the total by the tipping-bucket

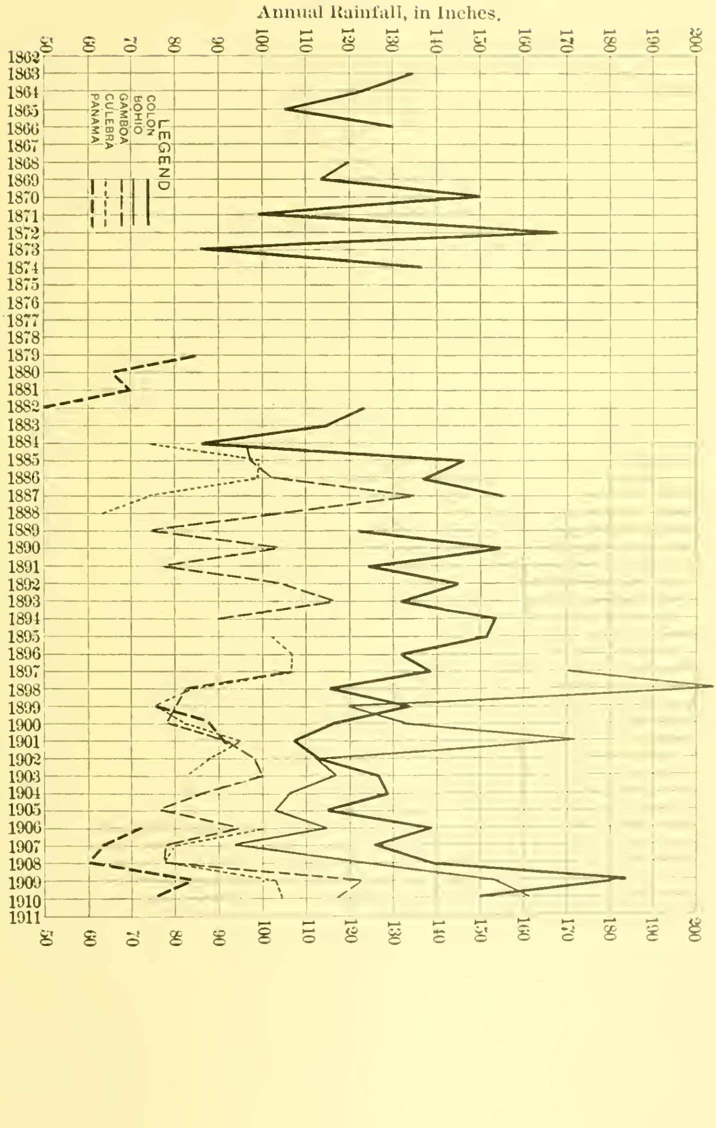

Annual Rainfall, in Inches.
LEGEND
COLON
BOHIO
GAMBOA
CULEBRA
PANAMA

Fig. 13.

gauge. It appears that the records from these two gauges agreed about as closely as would the records of two gauges of the same type if exposed side by side.

Like all automatic devices, however, the records are only reliable when the instruments receive proper care and attention, and when their peculiarities are known and accounted for. Some investigation was made of the relation between the quantity recorded by the register

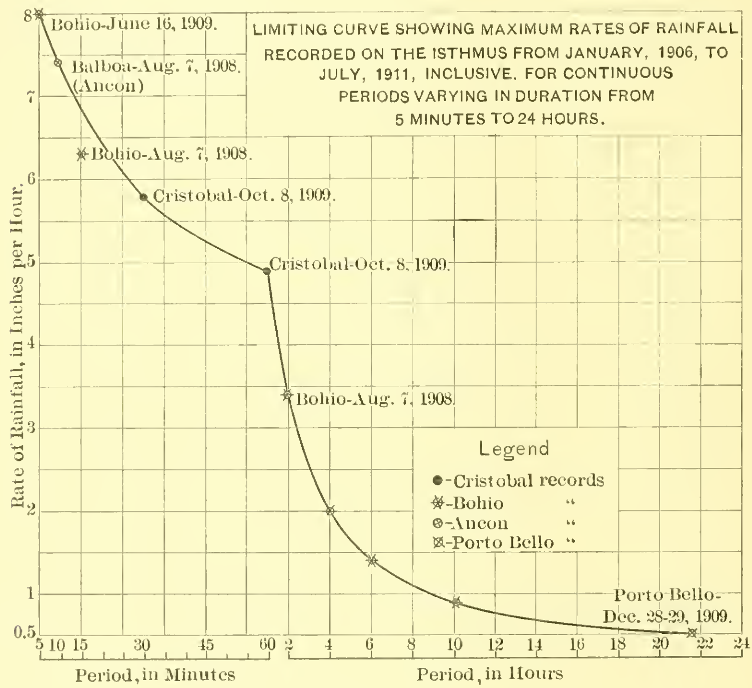


FIG. 14.

and the actual collection as indicated by stick measurement. It was found that the ratio depended largely on the quantity and character of the rainfall. For light and slow rains, the automatic registration accorded closely with the stick measurement, but for heavy rains, a marked deficiency of record appeared with the automatic registers. This loss was due to an appreciable quantity of water which pours into the tipped bucket while it is tipping. Each bucket, therefore, is over-filled, and, as only $\frac{1}{100}$ in. is recorded for each tip, all the water

flowing down the funnel is not measured. The deficiency in the automatic record was found to vary from less than 1% to as much as 8 or 10%, in exceptional cases. There are other factors, also, which contribute to defective registration, among which are lack of care in leveling up the gauge when setting, insufficient lubrication of the moving parts, weak battery power, and poor contacts in the electrical connections. The records in Table 8 are given as an indication of the foregoing conditions.

TABLE 8.

BEST CONDITIONS.

Date.	Gauge.	Gauge record.	Stick measurement.	Percentage registered.
April 25th, 1911	Culebra automatic	2.77 in.	2.84 in.	98
May 3d, 1911	" "	1.18 "	1.19 "	99
May 9th, 1911	" "	1.88 "	1.93 "	97
May 11th, 1911	" "	2.21 "	2.24 "	99
April 25th	Rio Grande automatic	3.12 "	3.12 "	100
May 3d	" " "	1.08 "	1.08 "	100
May 9th	" " "	2.06 "	2.08 "	99
May 11th	" " "	2.92 "	2.97 "	98

WORST CONDITIONS.

Date.	Gauge.	Gauge record.	Stick measurement.	Percentage registered.
April 4th, 1911	Balboa automatic	1.23 in.	1.28 in.	96
April 24th, 25th	" "	2.56 "	2.65 "	97
April 30th	" "	1.56 "	1.75 "	89
May 11th	" "	1.49 "	1.60 "	93
April 24th, 1911	Empire automatic	2.02 "	2.28 "	89
May 3d	" "	0.47 "	0.54 "	89
May 9th	" "	1.86 "	2.07 "	90
May 11th	" "	2.28 "	2.57 "	89

The trouble in the Balboa and Empire gauges was investigated and removed. In the latter case it was due to improper adjustment of the ratchet wheel; in the former it was due to defective leveling. These errors are not the fault of the instrument, but carelessness, inattention, or ignorance would fail to detect them, and the records would be misleading. On this account the total quantity of rain caught by the gauge is always measured by stick, and this reading is used in the records. The automatic record is used only to find the time the shower began and ended, and determine the rates of downpour during various periods.

As it was desirable to get rainfall records from isolated locations, where daily attention was not practicable and where the expense of a long-period gauge was not warranted, some investigation was made

as to the loss by evaporation from a standard rain gauge if the readings were taken at intervals of 2 weeks or longer. During November, 1910, the rainfall was allowed to accumulate in the standard rain gauge at Culebra, measurements being made on the 15th and 30th of the month. The total monthly rainfall by standard gauge showed a quantity slightly in excess of the total by the tipping-bucket gauge, which was close by, the collections in which were measured daily by stick before dumping. In other words, the nightly accumulation from dew and fog in the standard gauge more than equalled the evaporation during the day. It seems, therefore, that semi-monthly measurements of the accumulation in a standard rain gauge during the rainy season will give the total monthly rainfall with a fair degree of accuracy. On account of the construction of the standard rain gauge, its contents are well insulated from the effects of solar radiation until the inner tube is filled, the capacity of the latter being equivalent to 2 in. of rainfall, and, therefore, it seems that fairly reliable results may be obtained during the dry season by visiting the gauges only if necessary two or three times a month.

In regard to the collection from dew and fog, an appreciable quantity of moisture is gathered by night over the receiving surface of a rain gauge. This, however, enters only occasionally into monthly tables, because, if there has been no rain during the night, the moisture which has gathered in the automatic register is thrown out by the observer in the morning unmeasured. The only time this quantity does appear is when rain has fallen, and then it is impossible to separate the precipitation from the two sources. The nightly accumulations from fog and dew are greatest during foggy and clear nights and least during cloudy nights. The quantities range from zero to as much as 0.01 in. for a night. The average for the rainy season is estimated to be about 0.005 in. per night, or 0.15 in. per month. This, of course, does not mean that there is an equivalent of 0.15 in. of rainfall over the whole land surface from fog and dew; it merely means that the thin metal surface of the rain gauge has cooled rapidly by outward radiation, that the warmer air coming in contact therewith has been suddenly cooled, and the vapor condensed and precipitated on the surface of the gauge.

In order to get information concerning rainfall conditions at the heads of streams, three 14-day automatic rain gauges, manufactured in

Holland, were purchased. These gauges have now been in operation more than one year, and the following notes are submitted regarding their value in securing automatic rainfall records at interior points which cannot be visited except at infrequent intervals.

The clocks which actuate the drums on these registers were designed to run 14 days without rewinding, but experience has shown that they cannot be depended on in this climate for periods of more than 10 days. Thus it has been found necessary to visit these gauges at least three times a month.

The drums are driven at such a slow speed that during heavy rains, so frequent in tropical countries such as the Canal Zone, the bucket tips so fast that the record is blurred and the individual tips cannot be counted. The blurred and indistinct character of the record makes it always difficult and at times impossible to determine accurately the quantity of rain falling during heavy showers.

Certain parts of the registering apparatus have been found to get out of order readily, and various parts deteriorate rapidly from rust and other causes when exposed to the climatic conditions obtaining on the Isthmus. When parts of these gauges become worn or broken, it is very difficult, if not impossible, to secure new parts to replace them, as neither the gauge nor any of its parts are manufactured in the United States. If, however, a standard gauge is placed by the side of the 14-day automatic gauge, as a check, it is possible to get daily automatic records of dependable accuracy.

Long-continued rain storms extending over a large area are of infrequent occurrence in the Canal Zone. The rainfall is usually in the form of showers of limited extent, and it has been found that the topography and the location of the rain gauge are most important factors in getting correct results. To investigate this matter somewhat, records from a set of four gauges located in Empire were compared. These gauges are placed almost in a straight line northeast and southwest, and across the valleys of the Obispo and Camacho Rivers from hill to hill. No. 1 is on the east side of the Canal not far from the Empire suspension bridge, at about Elevation 190. No. 2 is the permanent gauge from which the official records are taken, and is about 1 000 ft. southwest of No. 1, and on top of a hill, Elevation 330, on the west side of the Canal, not far from the Division Office. No. 3 is about 2 600 ft. southwest of No. 2, and is in the corral yard, Elevation 220,

a most excellent exposure. No. 4 is at Camacho Reservoir, Elevation 370, and is about 4 600 ft. southwest of No. 3. Gauges Nos. 1 and 3 are in the valleys, while Nos. 2 and 4 are on the hills, but the extreme difference in elevation is not more than 200 ft. Table 9 shows the records, in inches per month, obtained from these gauges:

TABLE 9.—Records of Rain Gauges at Empire.

Month.	No. 1.	No. 2.	No. 3.	No. 4.
June	7.98	5.98	6.43	4.62
July	6.44	4.00	5.94	6.62
August	6.96	5.98	8.50	7.98
September	5.97	5.46	6.20	6.97
October	15.37	14.97	18.05	18.81

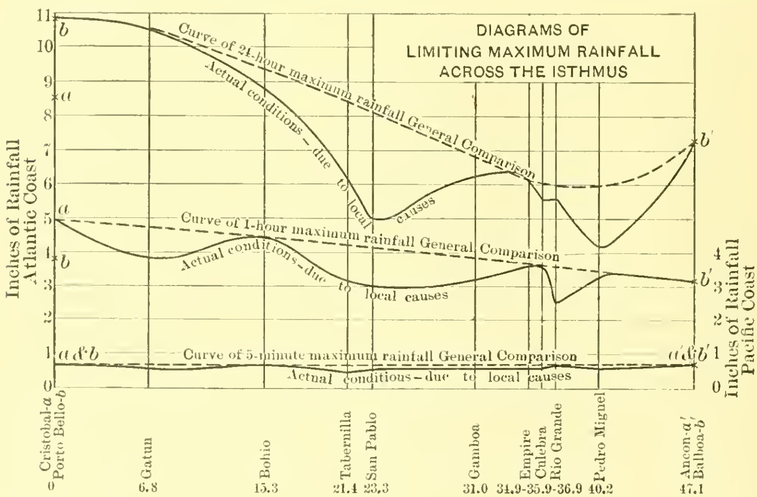


Fig. 15.

From inspection, it seems possible that the annual rainfall hitherto recorded at Empire may be from 12 to 15% too low. Table 10 gives information of a similar character, but mostly illustrating the limited extent of some of these tropical downpours. The records were obtained from Gatun and stations in the vicinity during a heavy rain on May 23d, the distances given being in an air line from Gatun Evaporation Station No. 2. (Gatun Evaporation Stations Nos. 1, 2, and 3 are located on the surface of the water in Gatun Lake.)

TABLE 10.

Station.	Rainfall, in inches.	Air line distance, in miles.	Direction.
Gatun Evaporation Station No. 2.	4.05	0.00	Origin.
" " " " 3.	3.36	0.65	S. E.
" " " " 1.	1.70	1.20	N. W.
Gatun Spillway	0.89	1.70	N. W.
Monte Lirio	3.75	2.90	N. E.
Brazos Brook	1.90	5.00	N. E.
Bohio	3.75	5.90	S. E.
Cristobal	2.42	6.90	North.

The diagram, Fig. 16, shows the frequency of heavy downpour on both sides of the Isthmus. It will be seen that the heaviest rains occur on the Atlantic side, and that there, too, are to be found the least number of days without any rain at all. This diagram represents average conditions.

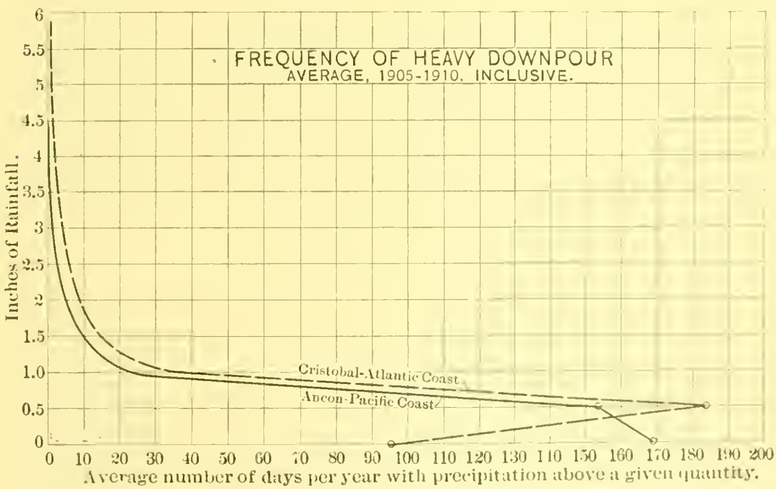


FIG. 16.

Storms giving a greater precipitation than 6.00 in. on the Atlantic or 4.50 in. on the Pacific side in 24 hours are of infrequent occurrence. During the past five years (June, 1906, to July, 1911), there have been 78 storms in which the precipitation has exceeded 4.00 in. Of these, 41 have had a precipitation of not more than 5.00 in., and 70 not more than 7.50 in. The rainfall in each of the remaining storms was: one not more than 8.00 in., one not more than 8.50 in., four not more

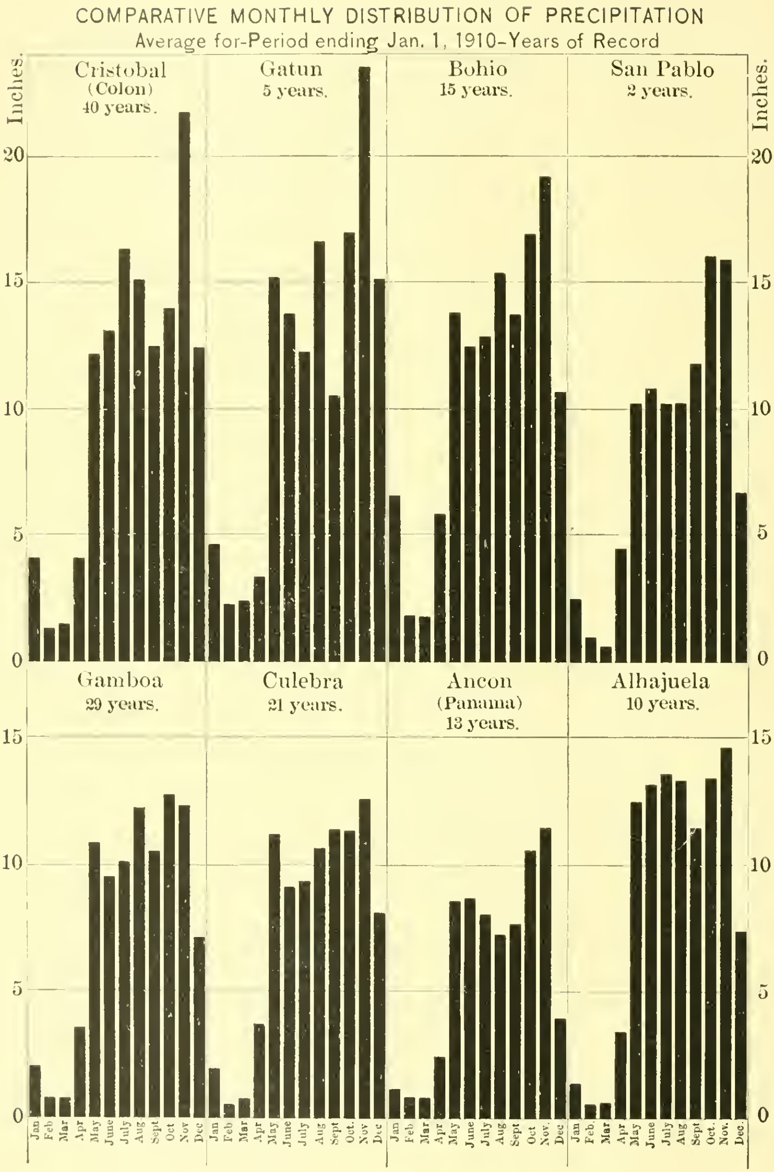

COMPARATIVE MONTHLY DISTRIBUTION OF PRECIPITATION
Average for-Period ending Jan. 1, 1910-Years of Record
Inches.
Cristobal
(Colon)
40 years.
Gatun
5 years.
Bohio
15 years.
San Pablo
2 years.
Gamboa
29 years.
Culebra
21 years.
Ancon
(Panama)
13 years.
Alhajuela
10 years.
0
5
10
15
20
Jan Feb Mar Apr May June July Aug Sept Oct Nov Dec

FIG. 17.

than 9.00 in., and two not more than 11.00 in. During this period the rainfall on the Pacific side has exceeded 4.50 in. at one or more stations on six different dates, or an average of a little more than once each year. On the Atlantic side the rainfall has exceeded 6.00 in. at one or more stations on fifteen different dates, or an average of about three times per year. It appears, therefore, that a rainfall of 6 in. in 24 hours is of not infrequent occurrence during the year, and that twice in 5 years a downpour of from 10.50 to 11 in. may be expected. Fig. 14 is a graphical representation of the extreme maximum limits of rainfall during the years of record.

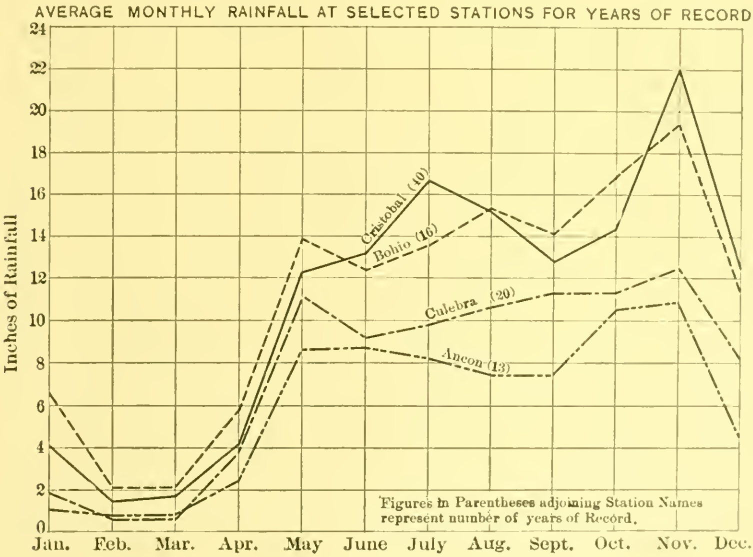


FIG. 18.

Fig. 18 shows the average monthly rainfall at four stations having long-time records. These stations indicate such conditions as are found from ocean to ocean across the Isthmus, and show that even in the so-called dry season there is an appreciable quantity of rainfall. The diagrams, Figs. 19 and 20, show, respectively, the maximum and minimum monthly precipitation for the years of record for the stations mentioned above. The greatest quantity recorded in the Zone was at Colon in November, 1862. Quantities exceeding this, however, were recorded at Porto Bello in 1909, where 45.03 in. fell in November and

58.17 in. in December. The total annual rainfall for this station in 1909 was 237.28 in., which is the maximum of record on the Isthmus.

TABLE 11.—MAXIMUM RAINFALL IN THE CANAL ZONE, OCTOBER 1ST, 1905, TO JUNE 30TH, 1911.

Stations.	Maximum Rainfall.					
	5 Minutes.		1 Hour.		24 Hours.†	
	Inches.	Date.	Inches.	Date.	Inches.	Date.
Ancon, (October 1st, 1905)....	0.64	Aug. 7, 1908.	2.89	Aug. 7, 1908.	6.37	Nov. 16-17, 1906.
Balboa. (June 10th, 1906)......	0.63	Aug. 7. 1908.	5.86	June 2, 1906.	7.57	Nov. 16-17, 1906.
Pedro Miguel. (January 1st. 1908)....	0.60	Nov. 11, 1908.	3.30	Aug. 27, 1908.	4.56	Sept. 30-Oct. 1. 1909.
Rio Grande, (December 29th. 1905).	0.75	July 24, 1908.	2.74	Apr. 25, 1911.	6.00	Dec. 2-3, 1906.
Culebra, (July 1st, 1906)........	0.64	May 2. 1908.	3.69	Oct. 16, 1907.	5.55	Dec. 3, 1906.*
Empire, (July 18th, 1906).......	0.60	July 25, 1906.	3.63	Oct. 1, 1909.	6.15	Dec. 3, 1906.*
Gamboa, (November 18th, 1905).	0.59	July 27, 1908.	3.32	May 11, 1911.	6.56	Dec. 2-3. 1906.
Alhajuela, (March 31st, 1907).....	0.60	July 20, 1909.	3.40	Dec. 28. 1909.	8.19	Dec. 3, 1906.*
San Pablo, (November 1st, 1907)..	0.60	Oct. 29. 1908.	3.10	Oct. 29, 1908.	5.14	Nov. 11-12, 1909.
Tabernilla. (November 1st, 1907)..	0.50	Oct. 31, 1909.	3.09	Aug. 18, 1908.	6.22	Sept. 4-5. 1910.
Bohio. (October 1st. 1905)....	0.67	June 16, 1909.	4.51	Aug. 7. 1908.	8.85	Aug 7-8. 1908.
Gatun, (August 24th, 1907)....	0.61	July 16. 1908.	3.82	May 26. 1910.	10.48	Dec. 3, 1906.*
Cristobal, (October 1st, 1905)....	0.64	Aug. 25. 1909	4.90	Oct. 8. 1909.	8.53	Dec. 2-3, 1906.
Porto Bello, (May 1st. 1908).........	0.64	Aug. 7. 1908.	3.77	Aug. 7, 1908	10.86	Dec. 28-29, 1909.

NOTE.—Dates in parentheses under station names refer to installation of automatic rainfall registers.

* No automatic record on these dates. Total for 24 hours ending at noon.

† Maximum rainfall for any 24 consecutive hours.

In Table 11 may be found the record of the maximum rainfalls in the Canal Zone, October 1st, 1905, to June 30th, 1911, for periods of 5 min., 1 hour, and 24 hours. The curves on Fig. 13 show the relative distribution of excessive precipitation with relation to geographical location. Table 12 gives the periods of maximum rainfall for Cristobal, Gamboa, and Bohio.

Table 13 is compiled from the reports of the U. S. Weather Bureau, and is given for the purpose of showing the mean monthly rainfall

MAXIMUM MONTHLY RAINFALL AT SELECTED STATIONS FOR YEARS OF RECORD

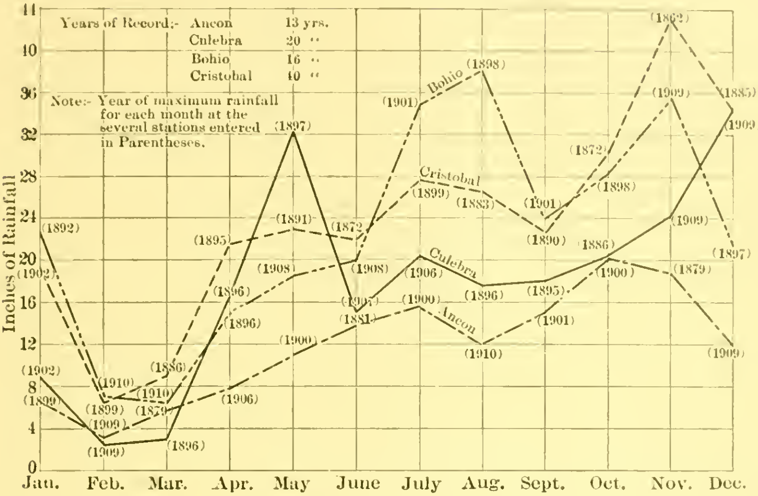


FIG. 19.

MINIMUM MONTHLY RAINFALL AT SELECTED STATIONS FOR YEARS OF RECORD

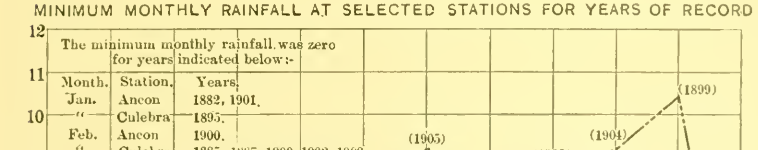


FIG. 20.

at selected stations covering geographically the whole of the United States. It is commonly believed that the rainfall on the Isthmus and in the tropics generally is very much heavier than in the United States and the temperate zone. There are several areas, however, in different parts of the United States, where the average annual rainfall equals or exceeds that on certain parts of the Isthmus, and this is especially noticeable when compared with the Ancon (Panama) records. The average and maximum annual rainfall records at a few selected stations are given in Table 14.

The line between the dry and rainy seasons is neither constant nor clearly marked. In some years the dry season begins as early as the middle of December, and in others not until toward the end of January. The dry season usually begins early on the Pacific side, and on the Atlantic side it has a tendency to lag considerably behind. This accounts for some of the freshets in the Chagres during January and February, when conditions in the interior do not seem to warrant their occurrence. During the first 6 months of the rainy season, the rain comes in heavy local showers, and the downpour is usually of short duration. In November and December, rains of a more general character occur, which cover wide areas and at times have a duration of 24 hours or more. The intensity of precipitation, however, is much less than that of the local showers of the preceding months.

Summarizing partly, from Table 11, "Maximum Rainfall in the Canal Zone," Table 15 shows the stations which recorded the extreme limits.

As has been stated previously, severe rainstorms of general distribution are not frequent on the Isthmus. The storm of December 2d and 3d, 1906, was the heaviest of record in the Canal Zone. This storm produced one of the greatest of the Chagres River freshets, and interrupted work for a very considerable period, besides bodily damaging the railroad and other works. The quantities recorded are given in Table 16.

Table 17 shows that, as a rule, the heavy downpours are of an extremely local character, and an examination of the records indicates this condition during showers of recent occurrence.

In connection with what has been stated regarding the quantity of rainfall, it may be interesting to note that the location of the heaviest known rainfall in the world is in the district of Assam,

TABLE 12.—PERIODS OF MINIMUM RAINFALL.
(Consecutive Periods—Calendar Months.)

Length of period.	CRISTOBAL. Amount, in inches.	CRISTOBAL. Period.	BOHIO. Amount, in inches.	BOHIO. Period.	GAMBOA. Amount, in inches.	GAMBOA. Period.
1 month..	0.04	February, 1897................	0.00	February, 1897....................	0.00	(3 months) Feb., 1891, Mch., 1897, Mch., 1898.
2 months.	0.32	February and March, 1897.....	0.25	February and March, 1897........	0.12	February and March, 1898.
3 " .	2.01	Jan. to Mar., 1885, inclusive...	2.14	Feb. to April, 1905, inclusive..	0.73	Jan. to March, 1908, inclusive.
4 " .	3.35	Jan. to Apr., 1885, " ...	5.13	Jan. " April, 1907, " ..	1.36	Jan. to April, 1907, "
5 " .	6.97	Dec., 1884, to Apr., 1885 " ...	9.57	Dec., 1907, to April, 1908 " ..	2.37	Dec., 1900, to April, 1901 "
6 " .	14.02	Nov., 1884, " Apr., 1885 " ...	20.95	Nov., 1907, " April, 1908 " ..	13.24	Dec., 1900, " May, 1901 "
7 " .	21.93	Nov., 1884, " May, 1885 " ...	31.35	Dec., 1902, " June, 1903 " ..	20.92	Dec., 1900, " June, 1901 "
8 " .	30.59	Oct., 1884, " May, 1885 " ...	42.05	Dec., 1902, " July, 1903 " ..	29.23	Nov., 1897, " June, 1898 "
9 " .	39.96	Sept., 1884, " May, 1885 " ...	53.09	Dec., 1902, " Aug., 1903 " ..	38.73	Dec., 1890, " Aug., 1891 "
10 " .	53.23	Aug., 1884, " May, 1885 " ...	66.16	Nov., 1902, " Aug., 1903 " ..	48.65	Nov., 1890, " Aug., 1891 "
11 " .	68.82	July, 1884, " May, 1885 " ...	77.44	June, 1904, " April, 1905 " ..	59.12	Nov., 1890, " Sept., 1891 "
1 year....	79.14	June, 1884, " May, 1885 " ...	89.68	May, 1904, " April, 1905 " ..	73.55	Dec., 1907, " Nov., 1908 "
2 years...	196.61	June, 1883, " May, 1885 " ...	191.36	June, 1904, " May, 1906 " ..	150.69	May, 1899, " April 1901 "
3 " ...	317.80	June, 1882, " May, 1885 " ...	297.72	June, 1904, " May, 1907 " ..	238.51	May, 1898, " April, 1901 "
4 " ...	422.83	June, 1881, " May, 1885 " ...	396.50	May, 1904, " April, 1908 " ..	320.63	Oct., 1897, " Sept., 1901 "
5 " ...	575.37	May, 1898, " April, 1903 " ...	520.76	June, 1902, " May, 1907 " ..	418.44	Feb., 1904, " Jan., 1909 "
6 " ...	707.37	Aug., 1898, " July, 1904 " ...	621.20	May, 1902, " April, 1908 " ..	513.81	Dec., 1902, " Nov., 1908 "
7 " ...	828.44	Aug., 1898, " July, 1905 " ...	747.81	Feb., 1902, " Jan., 1909 " ..	605.62	Feb., 1902, " Jan., 1909 "
8 " ...	942.91	Aug., 1898, " July, 1906 " ...	934.23	Jan., 1901, " Dec., 1908 " ..	697.28	Feb., 1898, " Jan., 1906 "
9 " ...	1 081.81	July, 1897, " June, 1906 " ...	1 052.76	Apr., 1899, " Mar., 1908 " ..	780.87	Nov., 1897, " Oct., 1906 "
10 " ...	1 209.03	June, 1896, " May, 1906 " ...	1 181.52	Mar., 1899, " Feb., 1909 " ..	861.92	Dec., 1898, " Nov., 1908 "
11 " ...	1 342.15	June, 1896, " May, 1907 " ...	1 383.89	Feb., 1898, " Jan., 1909 " ..	949.07	Jan., 1898, " Dec., 1908 "
12 " ...	1 480.41	May, 1896, " April, 1908 " ...	1 558.70	Jan., 1897, " Dec., 1908 " ..	1 056.40	Jan., 1897, " Dec., 1908 "
13 " ...	1 620.13	Nov., 1895, " Oct., 1908 " ...				
14 " ...	1 776.24	Jan., 1895, " Dec., 1908 " ...				
15 " ...	1 924 13	Jan., 1893, " Dec., 1907 " ...				
16 " ...	2 058 16	Aug., 1892, " July, 1908 " ...				
17 " ...	2 200 87	July, 1891, " June, 1908 " ...				
18 " ...	2 331.84	Jan., 1891, " Dec., 1908 " ...				
19 " ...	2 476.68	Nov., 1890, " Oct., 1909 " ...				
20 " ...	2 607.64	Jan., 1889, " Dec., 1908 " ...				

Note:—Length of rainfall records at Bohio, twelve years.

Note:—Gamboa rainfall records for 1895 and 1896 incomplete.

TABLE 13.—MEAN ANNUAL RAINFALL, IN INCHES, BY

DISTRICT.	1		2		3		4		5	
City.	Boston.	Philadelphia.	Norfolk.	Jacksonville.	Knoxville.	Cincinnati.	Cleveland.	Marquette.	Peoria.	St. Paul.
Years of Record.	96	39	38	56	38	74	51	38	57	72
January	3.74	3.23	3.30	2.82	4.78	3.27	2.52	2.08	1.77	0.89
February	3.51	3.35	3.78	3.25	4.75	3.18	2.61	1.73	2.04	0.79
March	4.14	3.43	4.35	3.39	5.29	3.77	2.95	2.06	2.72	1.44
April	3.81	2.92	3.31	2.70	4.47	3.20	2.73	2.13	3.14	2.41
May	3.66	3.30	4.13	3.93	3.74	3.96	3.78	3.20	3.88	3.44
June	3.14	3.27	4.12	5.64	4.27	4.20	3.95	3.57	3.78	4.10
July	3.51	4.14	5.91	6.37	4.07	3.77	3.80	3.06	3.98	3.53
August	4.15	4.69	6.05	6.60	4.03	3.61	3.09	2.74	3.14	3.47
September	3.44	3.36	4.01	8.16	2.86	2.79	3.75	3.61	3.60	3.39
October	3.71	3.01	3.75	4.60	2.54	2.59	2.96	3.08	2.24	1.99
November	4.11	3.11	2.76	2.21	3.51	3.19	3.22	2.72	2.31	1.38
December	3.78	3.07	3.42	2.86	4.04	3.38	2.74	2.44	2.15	0.97
Totals	44.70	40.88	49.29	52.53	48.35	40.91	38.10	32.42	34.75	27.80

British India. The portion of the district affected is comparatively small in area, and is in the foot-hills on the south slope of the Himalaya Mountains. The average annual rainfall there is about 475 in., and the total for individual years has exceeded 800 in. Practically all the

TABLE 14.—AVERAGE AND MAXIMUM ANNUAL RAINFALL AT A FEW SELECTED STATIONS.

Station.	Location.	Annual Rainfall for Years of Record, in Inches.	
		Average.	Maximum.
Cristobal	Canal Zone	130.03	183.41
Clearwater, Wash	Pacific Coast	128.45	151.56
Bowmans Dam, Cal	West Slope, Cascade Mountains	75.63	135.70
Astoria, Ore	Pacific Coast	75.35	108.30
Ancon	Canal Zone	71.67	91.42
Ashford, Wash	West Slope, Cascade Mountains	70.24	84.25
McKenzie Bridge, Ore	West Slope, Cascade Mountains	69.84	86.11
Moline, Fla	North Gulf Coast	66.19	77.27
Mobile, Ala	North Gulf Coast	61.80	91.18
Franklin, La	North Gulf Coast	60.92	79.51
Hatteras, N. C	South Atlantic Coast	60.21	102.04

Months, at Selected Stations in the United States.

6			7		8		9		10	11	12	13
Leavenworth.	Williston, N. D.	Helena.	New Orleans.	Fort Sill.	Galveston.	El Paso.	Prescott.	Denver.	Salt Lake City.	San Diego.	San Francisco.	Portland, Ore.
72	29	28	71	39	39	50	42	37	35	60	60	58
1.10	0.60	1.00	4.54	1.13	3.41	0.42	1.56	0.48	1.33	1.76	4.82	6.32
1.37	0.45	0.66	4.28	1.19	3.02	0.47	1.96	0.47	1.48	1.96	3.63	4.96
1.87	0.67	0.83	4.56	1.55	2.90	0.29	1.71	0.96	2.04	1.46	3.32	4.74
2.92	1.23	1.06	4.53	2.84	3.07	0.19	0.82	2.09	2.07	0.61	1.68	2.94
4.49	2.15	2.13	4.06	5.14	3.36	0.28	0.55	2.57	2.16	0.34	0.74	2.38
5.15	3.61	2.26	5.39	3.72	4.19	0.59	0.24	1.46	0.77	0.06	0.02	1.77
4.22	1.96	1.13	6.53	3.14	3.96	1.67	2.85	1.64	0.50	0.06	0.02	0.75
4.21	1.32	0.69	5.65	2.97	4.75	1.88	3.20	1.35	0.85	0.11	0.02	0.58
3.58	0.92	1.13	4.49	3.14	5.72	1.64	1.05	0.90	0.91	0.08	0.31	1.69
2.42	0.32	0.77	3.25	2.58	4.33	0.87	0.78	0.97	1.43	0.34	1.03	3.11
2.09	0.56	0.76	3.81	1.79	3.90	0.57	1.11	0.56	1.39	0.95	2.62	6.00
1.38	0.57	0.79	4.54	1.66	3.70	0.44	1.57	0.60	1.40	1.83	4.64	7.21
34.80	14.86	13.21	55.63	30.85	46.31	9.31	17.40	14.05	16.33	9.56	22.83	42.45

rain falls in the 6 months of the summer monsoon, May to October, inclusive. The maximum daily record for this section is reported as 39 in.

TABLE 15.—Extreme Limits of Rainfall.

Period.	Amount, in inches.	Station.	Date.
5 minutes	0.75	Rio Grande,	July, 1908.
1 hour	5.86	Balboa,	June, 1906.
24 hours	10.86	Porto Bello,	Dec., 1909.
1 month	58.17	" "	" 1909.
1 year	237.28	" "	Year, 1909.

TABLE 16.—Rainfall in Storm of December 2d and 3d, 1906.

Station.	Inches.	Station.	Inches.	Station.	Inches.
Ancon	3.18	Bohio	6.13	Empire	6.15
Rio Grande	5.58	Brazos Brook	8.96	Alhajuela	8.19
Camacho	5.53	Balboa	2.41	Gatun	10.48
Gamboa	6.29	Culebra	5.55	Cristobal	8.17

TABLE 17.—LOCAL CHARACTER OF HEAVY DOWNPOURS.

Station.	Rainfall.	Station.	Rainfall.	Distance between stations, in miles.	Date.
Cristobal..........	2.41	Gatun	0.01	6.4	May 6th, 1911
Gorgona............	5.45	San Pablo	1.27	3.5	May 13th, 1911
Gatun *............	4.05	Gatun	0.89	1.2	May 23d, 1911
Monte Lirio.......	6.76	Brazos Brook.....	0.97	6.0	Feb. 8th, 1910
Gamboa............	3.21	Empire............	0.52	4.0	July 3d, 1910
Cristobal..........	2.94	Brazos Brook.....	0.34	2.5	Oct. 5th, 1910
Rio Grande........	3.25	Culebra	1.62	0.8	Nov. 11th, 1910

* Evaporation Station No. 2.

RUN-OFF.

All the water that flows in a stream comes from the rainfall, but by no means all the rainfall finds its way into the stream. What proportion ultimately does appear depends on many conditions and combinations, the principal factors of which are elevation, topography, geology, and vegetation. The proportions of the rainfall ordinarily found running in northern streams may be seen in Table 18, abstracted from the Report of the Board of Public Works, Philadelphia, Pa.

TABLE 18.—AVERAGE ANNUAL YIELDS OF SUNDRY WATER-SHEDS TO OCTOBER 1ST, 1908.

Water-sheds.	Period covered, in years.	Area, in square miles.	Average rainfall, in inches.	Average rainfall flowing off, in inches.	Percentage flowing off.	Average yield, in cubic feet per second per square mile of drainage.	Average yield, in cubic feet per second per square mile of drainage area, for each inch of rainfall.
Perkiomen, at Frederick............	25	152.0	47.179	23.276	49.313	1.7104	0.0360
Neshaminy, below Forks...............	25	139.3	48.113	23.111	48.033	1.6979	0.0350
Tohickon.............	25	102.2	48.751	27.264	55.920	2.1963	0.0450
Schuylkill............	10	1 915.0	48.139	21.564	44.800	1.5842	0.0330
Sudbury, Mass.......	33	75.2	45.99	22.387	48.680	1.6451	0.0380

It appears that, in streams of the character of those in Table 18, from 45 to 55% of the rainfall is ordinarily collectable, and that the yield may be taken as from 1.5 to 2.2 cu. ft. per sec. per sq. mile of drainage area. In the case of the Isthmian streams, for the past 10 years, the ratio of average annual run-off to rainfall has been more than 60%, and the average annual yield per square mile of the Chagres

River has averaged between 5.75 and 6.06 cu. ft. per sec., with some monthly averages of nearly twice as much.

For the purpose of measuring the discharge of the rivers and streams, gauging stations have been maintained at Gatun, Bohio, Gamboa, and Alhajuela, all on the Chagres River proper. River stage stations have also been maintained at Lagartera, on the Trinidad River, and at Monte Lirio, on the Gatun River, where several gaugings per month are usually taken. Besides this, a hydrographer makes regular trips to the principal secondary streams for the purpose of gauging, the intention being to get measurements at different gauge heights. On account of back-water from the lake, the Trinidad and Gatun River stations were abandoned in April, 1911, and December, 1910, respectively, after records had been obtained for about 4 years at each place. Bohio was abandoned as a gauging station in October, 1910, but is still maintained as a river stage station. Automatic water stage registers are in operation at all the regular stations, and a continuous record of river heights has been obtained since gaugings were begun.

A meter-rating station is maintained at Bohio, and all meters used on the work are rated and adjusted as nearly as possible at regular intervals.

The regular field work is carried on by the hydrographers in charge of the several stations, and after being worked up, the data obtained are sent to the central office for checking and filing. Gaugings are made frequently, and at as many different elevations as possible, in order to check the discharge curves. This course is necessary on account of the continuously changing cross-sections of the river bed, due to the shifting gravel bars. Cable stations are maintained at Gatun, Gamboa, and Alhajuela, and the gauging is done from cars, except in very low water, when wading or a boat is resorted to. All other gaugings are made by wading, under ordinary conditions, and by estimate of quantity passing under bridges, in times of freshet.

At Gatun the entire run-off from the Chagres Basin has passed through the spillway since April, 1910, and gaugings have been made regularly. Up to the dry season of 1911, and before the installation of the cable, gaugings were made from a boat. During periods of extremely low water, it was possible to work in the spillway itself, where, on account of the even flow and the shape of the concrete sec-

tion, very good measurements of the minimum discharge of the stream in 1911 were obtained. At the same time simultaneous gaugings were made at the cable station below, and from comparison with the spillway gaugings, it appeared that the results at the regular station were satisfactory.

Tables 19 to 30, inclusive, give the discharge measurements in various units for the Chagres River at Alhajuela, Gamboa, Bohio, and Gatun. Unless otherwise noted, the quantities are from actual gaugings—floats previous to 1907, and current meter work subsequently.

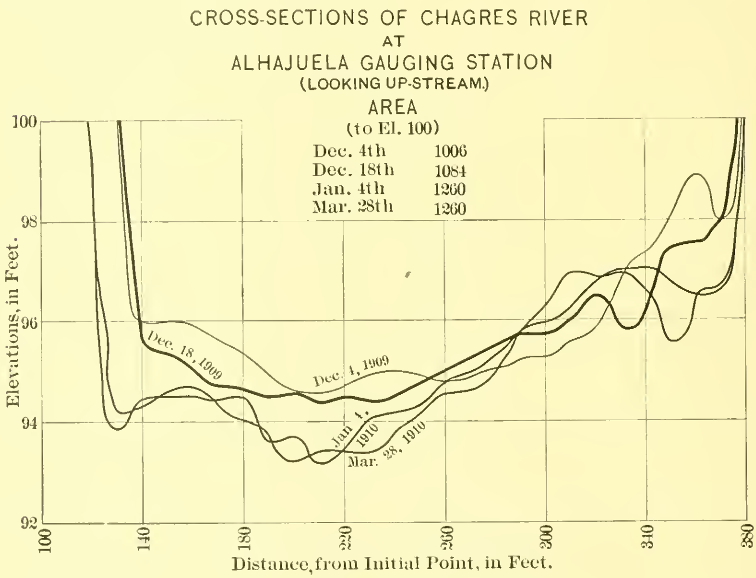


FIG. 21.

The discharge measurements for the stations on the Trinidad and Gatun Rivers for the years of record are given in Tables 31 and 32.

The Chagres River above Bohio has a constantly changing bed, and in order to get reliable measurements and estimates of the flow it is necessary to be constantly on the watch for changes in cross-section. After every freshet there is a change, and oftentimes of a most remarkable character. Some idea of this condition, perhaps, may be found from the fact that from one bar the Panama Railroad dredged some 60 000 cu. yd. of gravel in 1909, 90 000 cu. yd. from the same place in 1910, and about 95 000 cu. yd. in 1911. This gravel is dredged and stored during the dry season to be used as ballast on the new

railroad location. In spite of the removal, the pit is filled again during early flood season by most excellent gravel of various sizes, and, of course, cleanly washed. The cross-sections shown by Figs. 21 to 24, inclusive, were taken at the several stations, as required by the gauging work, and show the variations in area due to freshet scour. This action is perhaps best illustrated by comparing the cross-sections taken on December 18th, 1909, and January 4th, 1910, before and after one of the largest freshets recorded on this river. This rise began at Alhajuela on December 26th, and in the 9 days intervening, up to Jan-

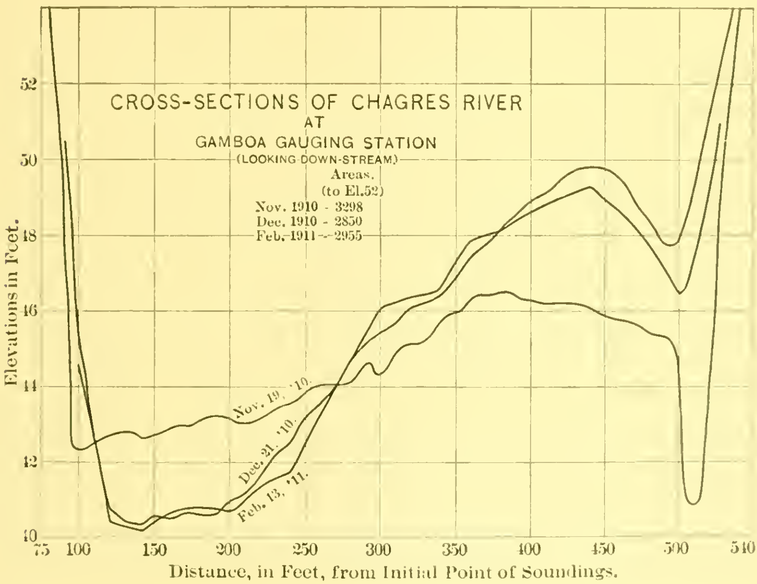


FIG. 22.

uary 4th, when the cross-section was taken, there had been a change in area of 176 sq. ft. below Elevation 100. The Gamboa station was not in operation at this time, but between December 21st, 1910, and February 13th, 1911, there was an increase of 95 sq. ft. in section at this station, due to a smaller freshet which caused high-water conditions for about 12 hours. The relation of change in cross-section is illustrated by the curves of area, velocity, and discharge for Gamboa shown by Fig. 25. These curves are made up from actual gaugings, and the change in cross-section producing the change in discharge can be seen by inspection of Fig. 22.

TABLE 19.—DISCHARGE, ALHAJUELA, IN CUBIC FEET PER SECOND.
Water-shed Area = 427 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890............	2 140*	1 095G	918G	883G	3 249G	4 061G	4 097G	4 309G	5 191G	4 909G	3 002G	4 485G	3 195
1891............	1 554G	777G	491G	424G	2 649G	1 801G	2 861G	2 119G	2 296G	3 390G	5 686G	3 885G	2 328
1892............	1 165G	918G	812G	1 695G	4 203G	2 649G	5 651G	4 626G	3 320G	4 521G	5 509G	4 627G	3 308
1893............	1 731G	1 660G	1 024G	2 402G	2 331G	2 437G	2 719G	3 602G	2 967G	2 860G	3 461G	6 110G	2 775
1894............	2 825G	1 342G	777G	600G	1 059G	1 695G	3 390G	2 967G	3 390G	3 567G	4 980G	8 088G	2 890
1895............	2 684G	1 236G	742M	918G	1 483M	2 048M	1 801M	2 543M	1 907M	3 638M	2 543M	3 920M	2 122
1896............	2 437M	1 059M	600M	1 271M	2 543M	2 225M	2 225G	2 154M	2 154M	2 048M	2 472M	3 885G	2 089
1897............	1 342G	918G	424G	563G	1 507G	1 801G	2 861G	3 108G	2 931G	2 931G	3 390G	3 532G	2 192
1898............	3 284G	1 236G	849G	1 519G	1 872G	2 190G	3 214G	3 143G	1 907G	2 649G	3 955G	1 731G	2 296
1899............	2 684G	777G	777G	883F	1 589F	2 154F	2 225F	3 284F	2 402F	2 649F	2 931F	2 260F	2 051
1900............	1 589F	812F	530F	565F	342F	1 554F	2 190F	2 825F	2 154F	3 214F	2 967F	2 296F	1 837
1901............	989F	671F	459F	459F	1 554F	1 731F	1 978F	2 437F	2 931F	2 755F	4 768F	3 037F	1 981
1902............	4 238	1 271	812	1 695	2 119*	2 190	2 119	2 331	2 366	3 214	3 143	1 836	2 278
1903............	1 130	848	194	565	1 377	1 942	3 108	3 178	2 790	2 755	4 009	5 474	2 306
1904............	3 002	1 589	954	2 190	1 820	2 659	2 677	1 579	2 834	2 031	4 143	2 350	2 319
1905............	1 418	777	578	442	1 932	1 594	1 248	2 207	1 722	2 688	2 267	1 265	1 512
1906............	682	613	434	711	1 098	1 363	3 019	3 242	2 783	2 080	3 433	7 283	2 228
1907............	3 995	1 376	1 013	652	2 132	2 989	4 225	2 955	3 904	4 583	3 866	2 896	2 882
1908............	1 005	582	585	592	2 595	2 213	2 639	3 007	3 710	3 611	6 400	4 123	2 588
1909............	4 040	3 315	1 155	1 110	2 420	4 870	3 300	4 090	3 560	3 665	11 300	17 300	5 010
Mean 20 Years..	2 197	1 144	722	1 007	2 094	2 308	2 877	2 985	2 861	3 188	4 211	1 519	2 509
1910............	5 050	2 870	1 625	3 140	5 220	3 310	4 420	4 400	4 339	4 398	4 490	5 964	4 102
Mean 21 Years..	2 333	1 226	765	1 109	2 243	2 356	2 951	3 053	2 932	3 246	4 225	4 588	2 585
1911............	1 640	2 310	881	1 689	2 790	2 881	2 609	.....	.....	.....	.....	.....	....
Mean 22 Years..	2 301	1 275	770	1 134	2 267	2 379	2 935	.....	.....	.....	.....	.....	....

* Monthly means. *G* From Gamboa : Alhajuela ratio. *M* Mean of measured discharges. *F* From formulas.

TABLE 20.—Discharge, Alhajuela, in Cubic Feet per Second per Square Mile.
Water-shed Area = 427 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890............	5.02	2.56	2.15	2.07	7.61	9.51	9.59	10.09	12.16	11.50	7.03	10.49	7.48
1891............	3.64	1.82	1.16	0.99	6.20	4.22	6.70	4.96	5.38	7.94	13.32	9.10	5.45
1892............	2.73	2.15	1.90	3.97	9.84	6.20	13.23	10.84	7.78	10.59	12.91	10.83	7.75
1893............	4.05	3.89	2.40	5.62	5.46	5.70	6.36	8.44	6.95	6.70	8.11	14.31	6.50
1894............	6.62	3.14	1.82	1.41	2.48	3.97	7.94	6.95	7.91	8.25	11.67	18.94	6.77
1895............	6.29	2.90	1.74	2.15	3.47	4.79	4.22	5.96	4.47	8.52	5.96	9.18	4.97
1896............	5.70	2.48	1.41	2.98	5.96	5.21	5.21	5.05	5.05	4.79	5.79	9.10	4.90
1897............	3.14	2.15	0.99	1.32	5.87	4.22	6.70	7.28	6.86	6.86	7.94	8.27	5.13
1898............	7.69	2.90	1.99	3.56	4.39	5.13	7.53	7.36	4.47	6.20	9.26	1.05	5.38
1899............	6.29	1.82	1.82	2.07	3.72	5.05	5.21	7.69	5.62	6.20	6.86	5.29	4.80
1900............	3.72	1.90	1.24	1.32	3.14	3.64	5.13	6.61	5.05	7.52	6.95	5.38	4.30
1901............	2.32	1.57	1.08	1.07	3.64	4.05	4.63	5.71	6.86	6.45	11.16	7.11	4.64
1902............	9.93	2.98	1.90	3.97	4.96	5.13	4.96	5.46	5.54	7.53	7.36	4.30	5.34
1903............	2.65	1.99	1.16	1.32	3.23	4.52	7.28	7.44	6.53	6.45	9.39	12.82	5.40
1904............	7.03	3.72	2.23	5.13	4.26	6.22	6.27	3.70	6.64	4.76	9.70	5.50	5.43
1905............	3.32	1.82	1.35	1.04	4.52	3.73	2.92	5.17	4.03	6.29	5.31	2.96	3.54
1906............	1.60	1.41	1.02	1.67	2.57	3.19	7.07	7.59	6.52	4.87	8.04	17.06	5.22
1907............	9.35	3.22	2.37	1.53	5.00	7.00	9.90	6.92	9.14	10.74	9.05	6.78	6.75
1908............	2.35	1.36	1.37	1.39	6.08	5.18	6.18	7.04	8.69	8.46	14.99	9.66	6.06
1909............	9.46	7.76	2.71	2.60	5.67	11.41	7.73	9.58	8.34	8.58	26.48	40.52	11.74
Mean 20 Years..	5.14	2.68	1.69	2.36	4.90	5.40	6.74	6.99	6.70	7.47	9.86	10.58	5.88
1910............	11.83	6.72	3.81	7.36	12.22	7.75	10.36	10.31	10.16	10.30	10.51	13.97	9.61
Mean 21 Years..	5.46	2.87	1.79	2.60	5.25	5.52	6.91	7.15	6.87	7.60	9.90	10.74	6.06
1911............	3.84	5.41	2.06	3.96	6.53	6.75	6.11	....	....	....	....	....	....
Mean 22 Years..	5.39	2.99	1.80	2.66	5.31	5.57	6.87	....	....	....	....	....	....

TABLE 21.—DISCHARGE, ALHAJUELA, IN INCHES ON WATER-SHED.
Water-shed Area = 427 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890	5.788	2.666	2.479	2.309	8.773	10.610	11.056	11.629	13.566	13.260	7.843	12.098	102.077
1891	4.197	1.895	1.337	1.105	7.148	4.708	7.724	5.718	6.002	9.154	14.864	10.491	74.343
1892	3.147	2.319	2.190	4.429	11.344	6.917	15.253	12.498	8.680	12.209	14.401	12.486	105.873
1893	4.669	4.051	2.767	6.270	6.295	6.359	7.382	9.730	7.754	7.724	9.048	16.501	88.500
1894	7.632	3.270	2.098	1.573	2.859	4.429	9.154	8.013	8.859	9.627	13.017	21.834	92.365
1895	7.252	3.020	2.006	2.399	4.001	5.344	4.865	6.871	4.987	9.823	6.650	10.584	67.802
1896	6.583	2.675	1.626	3.325	6.871	5.813	6.007	5.822	5.634	5.522	6.460	10.491	66.829
1897	3.620	2.239	1.141	1.473	6.767	4.708	7.721	8.393	7.651	7.909	8.859	9.534	70.021
1898	8.866	3.020	2.294	3.972	5.061	5.724	8.681	8.485	4.987	7.148	10.331	4.669	73.238
1899	7.252	1.895	2.098	2.309	4.289	5.634	6.007	8.866	6.270	7.148	7.654	6.099	65.521
1900	4.289	1.979	1.430	1.473	3.620	4.061	5.914	7.621	5.634	9.781	7.754	6.208	58.659
1901	2.675	1.635	1.245	1.194	4.197	4.519	5.338	6.583	7.654	7.436	12.452	8.197	63.125
1902	11.448	3.103	2.190	4.429	5.718	5.724	5.718	6.295	6.181	8.681	8.212	4.957	72.656
1903	3.055	2.072	1.337	1.473	3.724	5.043	8.393	8.578	7.286	7.436	10.476	11.782	73.655
1904	8.105	4.012	2.571	5.724	4.911	6.940	7.229	4.266	7.408	5.488	10.822	6.341	73.817
1905	3.828	1.895	1.556	1.160	5.211	4.162	3.366	5.960	4.496	7.252	5.924	3.413	48.223
1906	1.845	1.500	1.176	1.863	2.963	3.559	8.151	8.750	7.274	5.615	8.970	19.666	71.332
1907	10.780	3.353	2.732	1.707	5.764	7.810	11.414	7.978	10.197	12.384	10.097	7.817	92.033
1908	2.709	1.467	1.579	1.551	7.010	5.779	7.125	8.116	9.695	9.753	16.728	11.137	82.649
1909	10.906	8.081	3.124	2.901	6.537	12.731	8.912	11.045	9.305	9.892	29.546	46.714	159.694
Mean 20 Years	5.932	2.807	1.949	2.632	5.653	6.029	7.768	8.061	7.476	8.607	11.005	12.201	80.120
1910	13.636	6.998	4.393	8.212	14.092	8.647	11.942	11.882	11.336	11.870	11.722	16.107	130.837
Mean 21 Years	6.299	3.007	2.065	2.898	6.055	6.153	7.967	8.243	7.660	8.762	11.040	12.386	82.535
1911	4.427	5.634	2.375	4.418	7.528	7.531	7.041	......	......	......	......	......	......
Mean 22 Years	6.214	3.126	2.079	2.967	6.122	6.216	7.924	......	.....	......	......	.....	......

TABLE 22.—DISCHARGE, GAMBOA, IN CUBIC FEET PER SECOND.

Water-shed Area = 559 Sq. Miles.

Year.	Jan.	Feb.	Mar	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec	Annual mean.
1890	2 755	1 342*m*	1 095*m*	1 059	4 273	5 333	5 403*m*	5 686*m*	6 816*m*	6 498*m*	3 956*m*	5 898*m*	4 176
1891	2 048*m*	954*m*	600*m*	530*m*	3 461*m*	2 366*m*	3 779*m*	2 790*m*	3 037*m*	4 450*m*	7 487*m*	5 121*m*	3 052
1892	1 519*m*	1 095*m*	989*m*	2 048*m*	5 509	3 496	7 416	6 075	4 379	5 923	7 240	6 675	4 311
1893	2 296	2 013	1 236	2 931	3 037	3 214	3 567	4 732	3 885	3 714	4 556	8 052	3 605
1894	3 708	625	954	742	1 377	2 225	4 450	3 885	4 450	4 697*m*	6 534*m*	10 630*m*	3 690
1895	3 532	1 483	1 095*m*	1 130	3 461	3 178	3 073	4 061	3 390	4 944	3 532	3 779	3 055
1896	2 225	1 130	636*m*	1 342*m*	4 450*m*	4 132	2 981	2 331*m*	2 719*m*	2 684*m*	3 249	5 121*m*	2 746
1897	1 760*m*	1 130	494*m*	671	3 284*m*	2 366*m*	3 779*m*	4 097*m*	3 850*m*	3 850	4 450	4 626	2 863
1898	4 309	1 483	1 024	1 836	2 472	2 861	4 238	4 132	2 543	3 461	5 192	2 296	2 987
1899	3 496	954	954	1 059	1 907	2 472	2 825	4 238	3 178	3 390	3 714	2 649	2 572
1900	1 695	812	530	565	1 342	1 942	3 355	3 567	3 073	4 520	4 061	2 684	2 316
1901	1 024	671	459	459	1 766	2 048	2 190	2 861	3 638	3 702	6 746	3 143	2 392
1902	1 227	1 589	1 024	1 942	2 896	2 896	3 002	2 790	2 931	4 415	4 732	2 190	2 970
1903	1 625	918	706	706	1 519	2 154	3 320	4 341	4 697	3 956	5 721	7 028	3 058
1904	4 132	2 402	1 695	2 861	2 275	3 073	3 271	2 142	3 822	2 974	4 561	2 711	2 993
1905	1 687	1 032	844	703	2 183	2 091	1 684	2 613	2 231	3 480	2 585	1 835	1 914
1906	1 020	783	610	849	1 916	1 949	3 835	4 137	3 781	2 782	5 252	7 756	2 889
1907	2 935	1 297	1 011	687	1 446	2 879	5 560	3 890	5 140	6 030	5 090	3 810	3 315
1908	1 230	710	710	720	3 410	2 910	3 470	3 960	4 880	4 750	8 420	5 820	3 416
1909	4 930	4 040	1 410	1 350	3 180	6 410	4 340	5 380	4 680	4 820	14 870	22 760	6 514
1910	6 160	3 500	1 980	3 830	6 870	4 360	5 820	5 790	5 710	5 790	5 652	8 078	5 295
Mean 21 Years..	2 824	1 427	955	1 331	2 954	3 064	3 872	3 976	3 944	4 327	5 601	5 812	3 341
1911	1 766	2 362	976	1 756	3 178	3 020	3 516	.....	.....	.....	.....	.....	.....
Mean 22 Years..	2 776	1 469	956	1 353	2 962	3 062	3 856	.....	.....	.....	.....	.....	.....

m—Denotes mean of all gaugings during month.

TABLE 23.—DISCHARGE, GAMBOA, IN CUBIC FEET PER SECOND PER SQUARE MILE.
Water-shed Area = 559 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	4.92	2.40	1.96	1.90	7.64	9.54	9.67	10.18	12.20	11.61	7.07	10.54	7.47
1891	3.66	1.71	1.07	0.95	6.19	4.23	6.76	4.99	5.43	7.96	13.40	9.16	5.46
1892	2.72	1.96	1.77	3.66	9.86	6.25	13.27	10.88	7.83	10.60	12.95	10.88	7.72
1893	4.11	3.60	2.21	5.24	5.43	5.75	6.38	8.47	6.95	6.70	8.15	14.40	6.45
1894	6.63	1.12	1.71	1.33	2.46	3.98	7.96	6.95	7.96	8.40	11.69	19.02	6.60
1895	6.32	2.65	1.96	2.02	6.20	5.68	5.50	7.27	6.06	8.84	6.32	6.76	5.47
1896	3.98	2.02	1.14	2.40	7.96	7.40	5.25	4.17	4.86	4.80	5.81	9.16	4.91
1897	3.15	2.02	0.88	1.20	5.87	4.23	6.76	7.32	6.88	6.88	7.96	8.27	5.12
1898	7.70	2.65	1.83	3.28	4.42	5.12	7.58	7.39	4.55	6.20	9.29	4.10	5.34
1899	6.25	1.71	1.71	1.89	3.41	4.42	5.05	7.58	5.68	6.06	6.70	4.74	4.60
1900	3.03	1.46	0.95	1.01	2.40	3.47	6.00	6.38	5.50	8.08	7.26	4.80	4.20
1901	1.83	1.20	0.82	0.82	3.16	3.66	3.92	5.12	6.50	6.62	12.07	5.62	4.28
1902	9.35	2.84	1.83	3.47	5.18	5.18	5.37	4.99	5.24	7.90	8.47	3.92	5.31
1903	2.91	1.65	1.27	1.27	2.72	3.85	5.94	7.77	8.40	7.08	10.23	12.57	5.47
1904	7.39	4.30	3.03	5.12	4.07	5.50	5.85	3.83	6.84	5.32	8.16	4.85	5.36
1905	3.02	1.84	1.51	1.26	3.91	3.74	3.01	4.67	3.99	6.23	4.62	3.28	3.42
1906	1.82	1.40	1.09	1.52	3.43	3.49	6.86	7.40	6.76	4.98	9.39	13.88	5.17
1907	5.25	2.32	1.81	1.23	2.59	5.15	9.95	6.96	9.20	10.78	9.10	6.82	5.93
1908	2.20	1.27	1.27	1.29	6.10	5.20	6.20	7.08	8.73	8.50	15.05	10.41	6.11
1909	8.82	7.22	2.52	2.42	5.69	11.46	7.76	9.62	8.37	8.62	26.60	40.71	11.65
1910	11.01	6.26	3.54	6.85	12.29	7.80	10.41	10.36	10.20	10.36	10.11	14.42	9.47
Mean 21 Years..	5.05	2.55	1.71	2.39	5.28	5.48	6.93	7.11	7.05	7.74	10.02	10.40	5.98
1911	3.16	4.22	1.75	3.14	5.68	5.40	6.29	....	....	....	....	....	....
Mean 22 Years..	4.96	2.63	1.71	2.42	5.30	5.48	6.90	....	....	....	....	....	....

TABLE 24.—DISCHARGE, GAMBOA, IN INCHES, ON WATER-SHED.

Water-shed Area = 559 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890	5.672	2.499	2.260	2.120	8.808	10.644	11.148	11.736	13.610	13.382	7.888	12.151	101.921
1891	4.220	1.781	1.234	1.060	7.136	4.719	7.794	5.753	6.058	9.177	14.950	10.560	74.142
1892	3.136	2.114	2.041	4.083	11.367	6.973	15.297	12.546	8.736	12.220	14.445	12.546	105.504
1893	4.738	3.749	2.548	5.846	6.260	6.415	7.355	9.765	7.754	7.724	9.093	16.600	87.847
1894	7.644	1.166	1.971	1.484	2.836	4.440	9.177	8.013	8.881	9.684	13.039	21.924	90.259
1895	7.286	2.759	2.260	2.254	7.148	6.337	6.341	8.382	6.761	10.192	7.051	7.794	74.565
1896	4.589	2.179	1.314	2.678	9.177	8.256	6.053	4.808	5.422	5.534	6.482	10.560	67.052
1897	3.632	2.103	1.015	1.339	6.767	4.719	7.794	8.439	7.676	7.932	8.881	9.534	69.831
1898	8.877	2.759	2.110	3.659	5.096	5.712	8.739	8.520	5.076	7.148	10.365	4.727	72.788
1899	7.206	1.781	1.971	2.109	3.931	4.931	5.822	8.739	6.337	6.987	7.475	5.465	62.754
1900	3.493	1.520	1.095	1.127	2.767	3.871	6.917	7.355	6.136	9.315	8.100	5.534	57.230
1901	2.110	1.250	0.945	0.915	3.643	4.083	4.519	5.903	7.252	7.632	13.467	6.479	58.198
1902	10.780	2.957	2.110	3.871	5.972	5.779	6.191	5.753	5.846	9.108	9.450	4.519	72.336
1903	3.355	1.718	1.464	1.417	3.136	4.295	6.848	8.958	9.372	8.162	11.413	14.494	74.632
1904	8.520	4.638	3.493	5.712	4.692	6.136	6.744	4.416	7.631	6.133	9.104	5.592	72.811
1905	3.482	1.916	1.741	1.406	4.508	4.173	3.470	5.384	4.452	7.183	5.155	3.781	46.651
1906	2.098	1.458	1.257	1.696	3.954	3.894	7.909	8.531	7.542	5.741	10.476	16.006	70.562
1907	6.053	2.416	2.087	1.372	2.986	5.746	11.470	8.024	10.264	12.428	10.153	7.863	80.862
1908	2.536	1.370	1.464	1.439	7.033	5.802	7.148	8.162	9.740	9.800	16.795	12.002	83.291
1909	10.168	7.518	2.935	2.700	6.560	12.786	8.946	11.091	9.338	9.938	29.680	46.932	158.562
1910	12.692	6.519	4.081	7.643	14.169	8.702	12.002	11.942	11.380	11.942	11.281	16.624	128.977
Mean 21 Years	5.823	2.675	1.970	2.663	6.093	6.115	7.985	8.201	7.870	8.922	11.178	11.985	81.480
1911	3.643	4.394	2.018	3.503	6.548	6.025	7.252						
Mean 22 Years	5.724	2.753	1.972	2.702	6.068	6.111	7.952						

TABLE 25.—DISCHARGE, BOHIO, IN CUBIC FEET PER SECOND.
Water-shed Area = 779 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1890	4 132*	1 554*G*	1 271*G*	1 236*G*	6 569*G*	8 193*G*	8 300*G*	8 759*G*	10 489*G*	9 995*G*	6 075*G*	9 076*G*	6 304
1891	2 684*M*	1 377*M*	600*M*	600*M*	5 333*G*	3 214*M*	4 803*M*	4 521*M*	4 661*G*	6 851*G*	11 514*G*	7 876*G*	4 503
1892	2 331*G*	1 271*G*	1 165*G*	2 366*G*	8 476*G*	5 439*G*	11 407*G*	9 359*G*	6 746*G*	9 112*G*	11 125*G*	9 359*G*	6 513
1893	3 532*G*	2 331*G*	1 448*G*	3 426*G*	4 662*G*	4 944*M*	7 628*M*	10 772*M*	7 664*M*	8 582*M*	11 443*M*	18 541*M*	7 081
1894	7 063*M*	1 801*M*	1 045*M*	777*M*	3 355*M*	4 273*M*	6 216*M*	6 498*M*	7 452*M*	9 359*M*	12 714*M*	12 573*M*	6 098
1895	4 556*M*	1 377*M*	1 059*M*	1 059*M*	4 167*M*	4 309*M*	4 591*M*	6 922*M*	5 792*M*	6 922*M*	6 075*M*	6 957*M*	4 483
1896	4 732*M*	1 554*M*	883*M*	1 695*M*	4 944*M*	4 203*M*	3 143*M*	3 496*M*	5 933*M*	6 357*M*	7 593*M*	6 039*M*	4 214
1897	2 048*M*	1 589*M*	706*M*	883*M*	7 982*M*	4 203*M*	5 121*M*	7 275*M*	6 887*M*	7 487*M*	6 957*M*	6 816*M*	4 830
1898	5 651*F*	1 695*F*	1 095*F*	2 013*F*	2 684*F*	3 355*F*	6 569*F*	5 298*F*	3 214*F*	5 050*F*	7 840*F*	2 861*F*	3 944
1899	3 850*F*	1 731*F*	989*F*	1 095*F*	2 048*F*	3 143*F*	3 956*F*	5 721*F*	4 238*F*	4 944*F*	5 296*F*	3 602*F*	3 384
1900	2 119*F*	1 059*F*	742*F*	706*F*	1 625*F*	2 861*F*	5 615*F*	5 262*F*	4 768*F*	6 993*F*	6 605*F*	3 744*F*	3 508
1901	1 483*F*	989*F*	706*F*	365*F*	2 296*F*	2 790*F*	3 178*F*	4 309*F*	5 792*F*	6 322*F*	13 244*F*	4 591*F*	3 855
1902	11 231*F*	2 084*F*	1 271*F*	2 154*F*	4 273*F*	3 178*F*	3 037*F*	3 390*F*	3 920*F*	5 933*F*	6 675*F*	3 002*F*	4 179
1903	1 625*F*	1 024*F*	636*F*	565*F*	1 519*F*	2 543*F*	4 026*F*	5 474*F*	5 686*F*	5 580*F*	8 335*F*	10 489*F*	3 958
1904	4 944*F*	2 331*F*	1 413*F*	3 638*F*	3 037	4 732	4 520	3 178	6 674	4 909	7 628	4 132	4 261
1905	1 942	989	706	600	2 825	2 649	1 907	3 884	3 955	6 957	4 732	2 860	2 834
1906	1 338	864	611	1 002	2 152	2 672	5 920	6 356	4 557	4 078	7 826	10 654	4 078
1907	3 339	1 396	1 002	674	1 854	3 658	4 662	3 952	5 664	7 842	5 443	3 358	3 570
1908	1 281	681	586	666	3 894	3 131	3 626	5 491	5 599	5 558	9 515	5 046	3 821
1909	4 390	3 580	1 325	1 185	3 225	7 010	5 630	6 600	5 550	8 230	21 380	19 800	7 325
1910	6 160	2 894	3 080	1 875	3 760	7 470	5 950	9 350	8 010	7 840	....	....	....
Mean 20 Years.	3 714	1 564	963	1 335	3 846	4 025	5 193	5 826	5 807	6 853	8 901	5 769	4 637
Mean 21 Years	3 830	1 627	1 064	1 361	3 842	4 189	5 229	5 994	5 912	6 900	....	....	....

* Monthly Mean. *G* Gamboa: Bohio ratio. *M* Mean of Measured Discharges. *F* From Formulas.

TABLE 26.—DISCHARGE. BOHIO, IN CUBIC FEET PER SECOND PER SQUARE MILE.
Water-shed Area = 779 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	5.31	2.00	1.63	1.59	8.43	10.51	10.65	11.23	13.47	12.82	7.80	11.64	8.09
1891	3.45	1.77	0.77	0.77	6.84	4.13	6.16	5.80	5.98	8.79	14.78	10.11	5.78
1892	2.99	1.63	1.50	3.04	10.88	6.98	14.65	12.01	8.66	11.69	14.28	12.01	8.36
1893	4.54	2.99	1.86	4.40	5.99	6.35	9.79	13.83	9.84	11.02	14.69	23.81	9.09
1894	9.07	2.31	1.34	1.00	4.31	5.48	7.98	8.34	9.56	12.01	16.32	16.14	7.82
1895	5.85	1.77	1.36	1.36	5.35	5.53	5.90	8.88	7.44	8.88	7.80	8.93	5.75
1896	6.08	2.00	1.13	2.18	6.35	5.40	4.04	4.49	7.62	8.16	9.75	7.75	5.41
1897	2.63	2.04	0.91	1.13	10.24	5.40	6.58	9.34	8.84	9.61	8.93	8.75	6.20
1898	7.26	2.18	1.41	2.59	3.45	4.31	8.44	6.80	4.13	6.48	10.07	3.67	5.07
1899	4.94	2.22	1.27	1.41	2.63	4.04	5.08	7.34	5.44	6.35	6.80	4.62	4.34
1900	2.72	1.36	0.95	0.91	2.09	3.67	7.20	6.75	6.12	8.98	8.48	4.80	4.50
1901	1.90	1.27	0.91	0.47	2.95	3.58	4.08	5.53	7.43	8.12	17.00	5.89	4.93
1902	14.42	2.68	1.63	2.77	5.49	4.08	3.90	4.35	5.03	7.61	8.57	3.85	5.36
1903	2.09	1.31	0.82	0.73	1.95	3.26	5.17	7.03	7.30	7.16	10.69	13.47	5.08
1904	6.34	2.99	1.81	4.67	3.90	6.08	5.80	4.08	8.57	6.30	9.79	5.30	5.47
1905	2.49	1.27	0.91	0.77	3.63	3.40	2.45	4.99	5.08	8.92	6.08	3.67	3.64
1906	1.72	1.11	0.78	1.29	2.76	3.43	7.60	8.16	7.00	5.24	10.04	13.68	5.23
1907	4.29	1.79	1.29	0.86	2.38	4.70	5.99	5.08	7.27	10.07	6.99	4.31	4.58
1908	1.64	0.87	0.75	0.86	5.00	4.02	4.65	7.05	7.19	7.13	12.21	6.48	4.82
1909	5.64	4.60	1.70	1.52	4.14	9.00	7.23	8.47	7.12	10.56	27.44	25.42	9.40
1910	7.91	3.72	3.95	2.41	4.83	9.59	7.64	12.00	10.28	10.07	....	....	....
Mean 20 Years	4.77	2.01	1.24	1.72	4.94	5.17	6.67	7.48	7.45	8.80	11.43	9.72	5.95
Mean 21 Years	4.92	2.09	1.37	1.75	4.93	5.38	6.71	7.69	7.59	8.86	....	....	....

TABLE 27.—DISCHARGE, BOHIO, IN INCHES, ON WATER-SHED.
Water-shed Area = 779 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890	6.122	2.083	1.879	1.774	9.719	11.722	12.280	12.946	15.027	14.782	8.702	13.418	110.454
1891	3.977	1.843	0.888	0.859	7.886	4.608	7.102	6.687	6.672	10.134	16.488	11.652	78.796
1892	3.447	1.758	1.729	3.392	12.546	7.788	16.890	13.842	9.662	13.478	15.928	13.842	114.302
1893	5.234	3.111	2.144	4.909	6.906	7.085	11.287	15.946	10.978	12.704	16.389	27.451	124.147
1894	10.457	2.405	1.545	1.116	4.967	6.114	9.200	9.615	10.666	13.842	18.212	18.608	106.749
1895	6.744	1.843	1.568	1.517	6.168	6.170	6.802	10.238	8.301	10.238	8.702	10.295	78.586
1896	7.010	2.157	1.303	2.432	7.321	6.025	4.658	5.176	8.502	9.408	10.878	8.935	73.805
1897	3.032	2.124	1.049	1.261	11.804	6.025	7.586	10.768	9.863	11.079	9.963	10.088	84.642
1898	8.370	2.270	1.626	2.890	3.977	4.809	9.730	7.840	4.608	7.471	11.237	4.231	69.059
1899	5.695	2.312	1.464	1.573	3.032	4.507	5.857	8.462	6.069	7.321	7.587	5.326	59.205
1900	3.136	1.416	1.095	1.015	2.410	4.095	8.301	7.782	6.828	10.353	9.461	5.534	61.426
1901	2.190	1.322	1.049	0.524	3.401	3.994	4.704	6.375	8.290	9.361	18.970	6.791	66.971
1902	16.624	2.791	1.879	3.090	6.329	4.552	4.496	5.015	5.612	8.773	9.562	4.439	73.162
1903	2.410	1.364	0.945	0.814	2.248	3.637	5.960	8.105	8.145	8.255	11.929	15.527	69.339
1904	7.309	3.225	2.087	5.210	4.496	6.783	6.687	4.704	9.562	7.263	10.923	6.110	74.359
1905	2.871	1.322	1.049	0.859	4.185	3.793	2.825	5.753	5.668	10.284	6.783	4.231	49.623
1906	1.983	1.156	0.899	1.439	3.182	3.827	8.762	9.408	7.810	6.041	11.204	15.768	71.479
1907	4.946	1.864	1.487	0.960	2.744	5.244	6.906	5.857	8.111	11.607	7.799	4.969	62.491
1908	1.891	0.938	0.865	0.360	5.764	4.485	5.361	8.128	8.022	8.220	13.621	7.471	65.726
1909	6.502	4.790	1.960	1.696	4.773	10.041	8.335	9.765	7.944	12.176	30.614	29.304	127.900
1910	9.119	3.874	4.551	2.689	5.568	10.700	8.808	13.830	11.468	11.607	......	......	......
Mean 20 Years	5.498	2.105	1.426	1.914	5.693	5.765	7.686	8.621	8.317	10.140	12.748	11.200	81.113
Mean 21 Years	5.670	2.189	1.574	1.951	5.687	6.000	7.741	8.869	8.467	10.209	......	......	......

TABLE 28.—Discharge, Gatun, in Cubic Feet per Second.

Water-shed Area = 1 320 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890	6 694	2 518	2 059	2 002	10 624	13 273	13 446	14 190	16 992	16 102	9 812	14 703	10 204
1891	4 348	2 231	972	972	8 640	5 207	7 781	7 324	7 551	11 099	18 651	12 759	7 295
1892	3 776	2 059	1 887	3 833	13 731	8 811	18 479	15 162	10 928	14 741	18 022	15 162	10 549
1893	5 722	3 776	2 346	5 550	7 552	8 009	12 357	17 450	12 416	13 903	18 538	30 037	11 471
1894	11 442	2 918	1 693	1 259	5 435	6 922	10 070	10 527	12 072	15 162	20 597	20 368	9 872
1895	7 381	2 231	1 716	1 716	6 750	6 981	7 437	11 214	9 383	11 214	9 842	11 270	7 261
1896	7 666	2 518	1 430	2 745	8 009	6 809	5 092	5 664	9 612	10 298	12 301	9 783	6 827
1897	3 318	2 574	1 144	1 430	12 931	6 809	8 296	11 786	11 157	12 129	11 271	11 042	7 824
1898	9 155	2 746	1 774	3 261	4 348	5 435	10 642	8 583	5 207	8 181	12 701	4 635	6 389
1899	6 237	2 804	1 602	1 774	3 318	5 092	6 409	9 268	6 866	8 009	8 580	5 835	5 483
1900	3 433	1 716	1 202	1 144	2 632	4 635	9 096	8 524	7 724	11 329	10 700	6 065	5 683
1901	2 402	1 602	1 144	915	3 720	4 520	5 148	6 981	9 383	10 242	21 455	7 437	6 246
1902	18 194	3 376	2 059	3 490	4 318	5 148	4 920	5 492	6 350	9 612	10 814	4 863	6 553
1903	2 632	1 659	1 030	915	2 461	4 120	6 522	8 868	9 211	9 034	13 503	16 992	6 412
1904	8 009	3 776	2 289	5 891	4 910	7 652	7 308	5 139	10 792	7 938	12 331	6 681	6 894
1905	3 140	1 599	1 142	970	4 568	4 283	3 083	6 283	6 395	11 250	7 652	4 624	4 582
1906	2 164	1 397	988	1 620	3 479	4 321	8 472	10 277	8 824	6 594	12 654	17 237	6 501
1907	5 399	2 257	1 620	1 090	2 997	5 915	7 135	5 941	8 999	13 479	9 299	5 602	5 811
1908	1 976	1 046	851	892	5 046	5 025	5 744	8 886	8 708	8 859	11 408	8 017	5 788
1909	6 505	5 260	2 150	2 215	4 215	9 880	9 920	10 980	9 910	12 590	28 470	25 800	10 658
1910	11 740	5 080	3 160	6 090	10 270	11 060	13 800	13 020	11 637	12 710	17 312	21 820	11 472
Mean 21 Years	6 254	2 626	1 631	2 370	6 188	6 662	8 627	9 598	9 529	11 165	14 236	12 416	7 608
1911	3 636	2 999	1 439	2 071	5 952	6 488	6 905	....	....	....	....	....	....
Mean 22 Years	6 135	2 643	1 622	2 357	6 177	6 651	8 548	....	....	....	....	....	....

Jan., 1890, to Apr., 1907, from (Bohio Discharge) × (1.62).
May, 1907, to Apr., 1908, from (Bohio + Trinidad + Gatun River) × (1.10).

TABLE 29.—DISCHARGE, GATUN, IN CUBIC FEET PER SECOND PER SQUARE MILE.

Water-shed Area = 1 320 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual mean.
1890............	5.07	1.91	1.56	1.52	8.05	10.06	10.19	10.74	12.88	12.20	7.46	11.14	7.73
1891............	3.29	1.69	0.74	0.74	6.54	3.95	5.90	5.55	5.72	8.40	14.13	9.67	5.52
1892............	2.86	1.56	1.43	2.90	10.40	6.68	14.00	11.49	8.28	11.17	13.66	11.48	7.99
1893............	4.34	2.86	1.78	4.20	5.72	6.07	9.36	13.22	9.41	10.53	14.05	22.72	8.68
1894............	8.67	2.21	1.28	0.95	4.12	5.25	7.63	7.98	9.14	11.49	15.61	15.43	7.48
1895............	5.59	1.69	1.30	1.30	5.11	5.29	5.64	8.49	7.11	8.49	7.46	8.54	5.50
1896............	5.81	1.91	1.08	2.08	6.07	5.16	3.86	4.29	7.28	7.80	9.32	7.42	5.17
1897............	2.51	1.95	0.87	1.08	9.80	5.16	6.28	8.93	8.45	9.19	8.54	8.36	5.93
1898............	6.93	2.08	1.34	2.47	3.29	4.12	8.06	6.50	3.95	6.20	9.62	3.51	4.84
1899............	4.72	2.12	1.21	1.34	2.51	3.86	4.86	7.02	5.20	6.07	6.50	4.42	4.15
1900............	2.60	1.30	0.91	0.87	2.00	3.51	6.89	6.46	5.86	8.58	8.10	4.60	4.31
1901............	1.82	1.21	0.87	0.69	2.82	3.43	3.90	5.29	7.11	7.76	16.26	5.64	4.73
1902............	13.78	2.56	1.56	2.64	3.27	3.90	3.73	4.16	4.81	7.28	8.19	3.69	4.96
1903............	1.99	1.26	0.78	0.69	1.86	3.12	4.94	6.72	6.98	6.84	10.23	12.88	4.86
1904............	6.07	2.86	1.73	4.46	3.72	5.80	5.54	3.89	8.18	6.02	9.34	5.06	5.22
1905............	2.38	1.21	0.87	0.73	3.46	3.25	2.34	4.76	4.85	8.52	5.80	3.50	3.47
1906............	1.64	1.06	0.75	1.23	2.64	3.27	6.42	7.78	6.69	5.00	9.58	13.07	4.93
1907............	4.09	1.71	1.23	0.83	2.27	4.48	5.40	4.50	6.82	10.21	7.04	4.24	4.40
1908............	1.50	0.79	0.64	0.68	3.82	3.81	4.35	6.73	6.60	6.72	10.91	6.08	4.38
1909............	4.93	3.98	1.63	1.68	3.19	7.48	7.52	8.32	7.51	9.54	21.58	19.55	8.07
1910............	8.90	3.85	2.39	4.61	7.78	8.38	10.45	9.86	8.82	9.63	13.11	16.53	8.69
Mean 21 Years..	4.74	1.98	1.24	1.80	1.69	5.05	6.54	7.27	7.22	8.46	10.78	9.41	5.76
1911............	2.75	2.27	1.09	1.57	4.51	4.92	5.23	....	....	....	....	....	....
Mean 22 Years..	4.65	2.00	1.23	1.78	4.68	5.04	6.48	....	....	...	....	....	....

TABLE 30.—Discharge, Gatun, in Inches, on Water-shed.

Water-shed Area = 1 320 Sq. Miles.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual total.
1890............	5.845	1.989	1.799	1.696	9.281	11.226	11.748	12.384	14.368	14.070	8.323	12.844	105.573
1891............	3.793	1.760	0.853	0.826	7.540	4.407	6.802	6.399	6.382	9.684	15.763	11.148	75.357
1892............	3.297	1.682	1.649	3.236	11.990	7.453	16.140	13.248	9.238	12.877	15.242	13.236	109.288
1893............	5.004	2.978	2.052	4.686	6.595	6.772	10.791	15.242	10.499	12.143	15.675	26.194	118.631
1894............	9.996	2.301	1.476	1.060	4.750	5.857	8.797	9.200	10.197	13.248	17.412	17.786	102.080
1895............	6.445	1.760	1.499	1.450	5.891	5.902	6.502	9.788	7.933	9.788	8.323	9.846	75.127
1896............	6.698	2.060	1.245	2.321	6.998	5.757	4.450	4.946	8.122	8.993	10.398	8.554	70.542
1897............	2.894	2.031	1.003	1.205	11.298	5.757	7.240	10.295	9.428	10.595	9.528	9.638	80.912
1898............	7.990	2.166	1.545	2.756	3.793	4.597	9.292	7.494	4.407	7.148	10.733	4.047	65.968
1899............	5.442	2.197	1.395	1.495	2.894	4.307	5.603	8.093	5.802	6.998	7.252	5.096	56.574
1900............	2.998	1.354	1.049	0.971	2.306	3.916	7.943	7.448	6.538	9.892	9.037	5.303	58.755
1901............	2.098	1.260	1.003	0.770	3.251	3.827	4.496	6.099	7.933	8.946	18.142	6.502	64.327
1902............	15.886	2.666	1.799	2.945	3.770	4.351	4.300	4.796	5.367	8.393	9.138	4.254	67.665
1903............	2.294	1.312	0.899	0.770	2.144	3.481	5.695	7.747	7.788	7.886	11.413	14.846	66.277
1904............	6.998	3.085	1.994	4.976	4.289	6.471	6.387	4.485	9.126	6.940	10.421	5.831	71.006
1905............	2.744	1.260	1.003	0.814	3.989	3.626	2.698	5.488	5.411	9.823	6.471	4.035	47.362
1906............	1.891	1.104	0.865	1.372	3.044	3.618	7.402	8.969	7.464	5.764	10.688	15.067	67.280
1907............	4.715	1.781	1.418	0.926	2.617	4.998	6.226	5.188	7.609	11.771	7.855	4.888	59.992
1908............	1.729	0.852	0.738	0.759	4.404	4.251	5.015	7.759	7.364	7.747	12.171	7.010	59.799
1909............	5.684	4.144	1.879	1.874	3.678	8.345	8.670	9.592	8.379	10.999	24.078	22.540	109.862
1910............	10.261	4.009	2.755	5.143	8.969	9.350	12.050	11.367	9.840	11.102	14.631	19.056	118.533
Mean 21 Years..	5.462	2.083	1.425	2.002	5.404	5.633	7.535	8.382	8.057	9.753	12.033	10.844	78.613
1911............	3.170	2.364	1.257	1.752	5.200	5.489	6.030	......	......	......	......	......	......
Mean 22 Years..	5.358	2.096	1.417	1.991	5.396	5.627	7.467	......	......	......	......	......	......

TABLE 31.—Discharge, Trinidad River.

(Above Lagartera Station, Water-shed Area = 314 Sq. Miles.)

Cubic Feet per Second.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1907	....	....	....	....	556	1 097	985	845	1 334	3 144	2 076	1 135	....
1908	356	186	131	105	946	1 118	1 027	1 939	1 853	1 825	2 359	1 631	1 123
1909	1 398	988	365	210	782	1 870	1 760	2 155	1 940	2 580	2 660	2 470	1 598
1910	1 940	840	565	1 250	1 880	1 980	2 000	1 500	1 900	2 000	2 500	2 200	1 713
1911	1 000	300	170	190	....	....	....	....	....	....	....	....	....
Means (4 Years)	1 174	578	398	439	1 041	1 516	1 443	1 610	1 757	2 387	2 399	1 859	1 376

Cubic Feet per Second per Square Mile.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1907	....	....	....	....	1.77	3.49	3.14	2.69	4.25	10.02	6.49	3.61	....
1908	1.13	0.59	0.42	0.33	3.01	3.56	3.27	6.18	5.90	5.81	7.51	5.19	3.58
1909	4.45	3.15	1.16	0.67	2.49	5.96	5.60	6.86	6.18	8.22	8.47	7.87	5.09
1910	6.18	2.67	1.80	3.98	5.99	6.31	6.37	4.78	6.05	6.37	7.96	7.01	5.46
1911	3.18	0.96	0.54	0.60	....	....	....	....	....	....	....	....	....
Means (4 Years)	3.74	1.84	0.98	1.40	3.32	4.83	4.60	5.13	5.60	7.60	7.61	5.92	4.38

Inches on Water-shed.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
1907	.....	.....	.....	.....	2.041	3.894	3.620	3.101	4.742	11.552	7.241	4.162	....
1908	1.303	0.636	0.484	0.368	3.470	3.972	3.770	7.125	6.583	6.698	8.379	5.982	48.771
1909	5.130	3.280	1.337	0.748	2.871	6.650	6.456	7.909	6.895	9.477	9.450	9.073	69.276
1910	7.125	2.780	2.075	4.440	6.906	7.040	7.344	5.511	6.750	7.344	8.881	8.082	74.278
1911	3.666	1.000	0.623	0.669	....	....	....	....	....	....	....	....	....
Means (4 Years)	4.306	1.924	1.130	1.556	3.822	5.389	5.298	5.912	6.242	8.768	8.488	6.825	59.660

Station removed to ¼ mile below junction of Siri and Trinidad branch on October 7th, 1910.

TABLE 32.—DISCHARGE, GATUN RIVER.

(Above Monte Lirio—old station. Water-shed Area = 127 Sq. Miles.)

CUBIC FEET PER SECOND.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1907	....	....	....	....	315	622	839	604	1 183	1 268	935	600	....
1908	159	84	57	40	533	460	708	788	675	983	1 821	627	578
1909	615	470	157	114	454	1 170	848	780	930	1 160	2 450	2 600	979
1910	805	440	270	550	950	1 140	1 400	1 300	1 160	1 290	1 600	2 500	1 117
Means (3 and 4 Years).	526	331	161	235	563	848	949	868	987	1 175	1 702	1 582	827

CUBIC FEET PER SECOND PER SQUARE MILE.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
1907	....	....	....	....	2.48	4.90	6.61	4.76	9.32	9.97	7.37	4.72	....
1908	1.25	0.66	0.45	0.32	4.20	3.62	5.58	6.21	5.32	7.74	14.34	4.94	4.55
1909	4.84	3.70	1.24	0.90	3.58	9.21	6.68	6.15	7.33	9.13	19.30	20.46	7.71
1910	6.34	3.47	2.12	4.33	7.48	8.97	11.02	10.23	9.13	10.15	12.60	19.68	8.79
Means (3 and 4 Years).	4.14	2.61	1.27	1.85	4.44	6.68	7.47	6.84	7.78	9.25	13.40	12.45	6.52

INCHES ON WATER-SHED.

													Total.
1907	.....	.....	.....	.....	2.859	5.467	7.621	5.488	10.398	11.494	8.223	5.442	....
1908	1.441	0.712	0.519	0.357	4.842	4.039	6.433	7.159	5.936	8.923	15.998	5.695	62.054
1909	5.580	3.853	1.430	1.004	4.127	10.276	7.701	7.090	8.178	10.526	21.530	23.586	104.881
1910	7.309	3.613	2.444	4.831	8.624	10.008	12.704	11.793	10.186	11.700	14.060	22.688	119.960
Means (3 and 4 Years).	4.777	2.726	1.464	2.064	5.113	7.448	8.615	7.882	8.674	10.661	14.953	14 353	88.730

At Bohio the conditions are very different from those in the upper river. Here the river bed has very little slope, and the material carried along by the flood is alluvium and fine sandy clay. Besides this, the conditions are greatly modified by the large quantity of material from the canal excavation, which is dumped directly into the river and carried down stream. This has resulted in filling the bed to a greater extent than would otherwise have been the case. The changes in cross-section due to the various causes are shown by Fig. 23, the maximum change being about 200 sq. ft. below Elevation + 3 during the period between May 24th and December 23d, 1909. Since April, 1910, back-water conditions, and consequent lessening of the current, have resulted in a comparatively rapid filling of the bed at this point. On account of construction work at Gatun, it has been rather difficult to find a permanent gauging section at this station. Although the conditions of bottom and flow were not ideal, the best location seemed to be in the spillway channel below the concreted portion. Here the bottom was rather rough, and a little back-water was found near shore on each side. The eddies, cross-currents, and waves, which were found farther up in the concreted portion, due to bridge piers and other obstructions, were absent at the site chosen, and this location seemed to have the most advantages. A profile of the bottom, taken on February 28th, 1911, and the horizontal velocity curves taken on February 14th, 21st, and 28th, at different elevations of the lake surface, are shown by Fig. 24.

During the 21 years of record and estimate, for which the discharge tables have been made up, Alhajuela, Gamboa, and Bohio, had the maximum discharge year in 1909, and Gatun in 1910. This condition was brought about by the fact that the rainfall and consequent run-off which took place in November and December, 1909, was especially large during the latter part of December, 1909. This caused an excess of flow at the upper stations for the last part of the calendar year, 1909, but much of the water was not delivered at Gatun until the following January, thus appearing to bring up the average of the calendar year, 1910. The river year, 1909-10, however, was the year of maximum run-off at all stations. The year of minimum run-off was 1905 at all stations, and the year of minimum dry-season run-off was 1908. The interrelation of these periods of discharge, as compared with each other and with the mean discharge for the period, is shown by Figs. 26 to 29, inclusive. From these it may be seen that the ratio

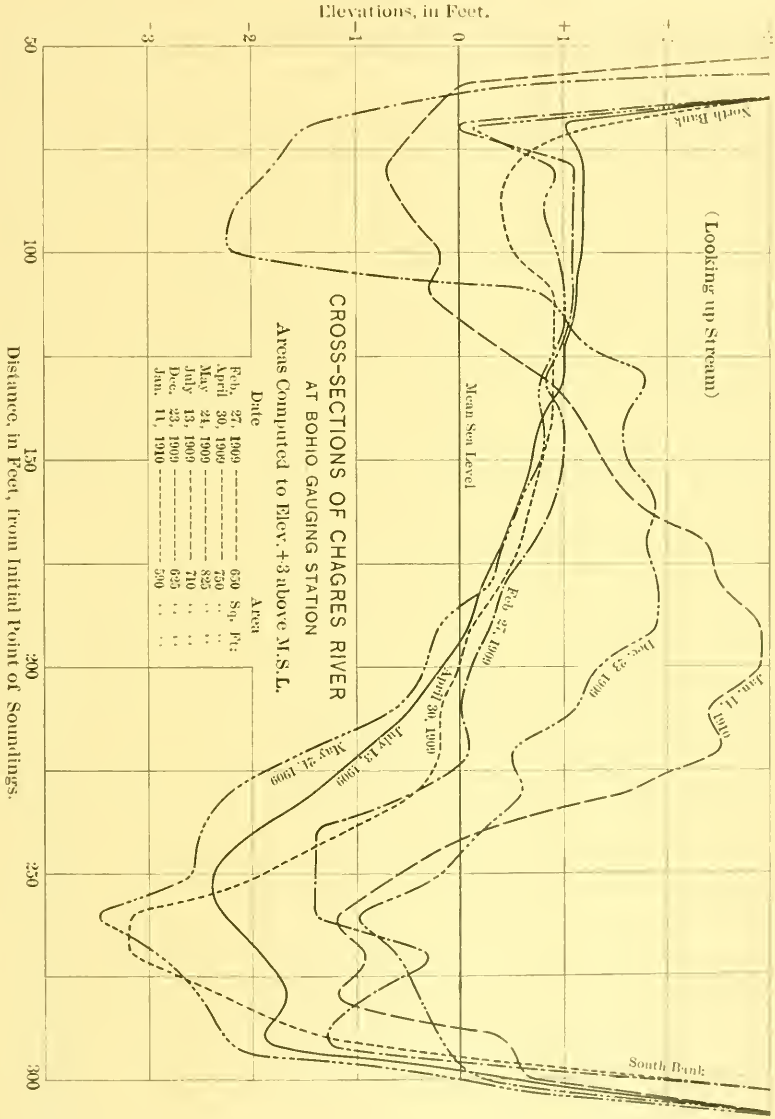

Elevations, in Feet.
(Looking up Stream)
North Bank
South Bank
Mean Sea Level
CROSS-SECTIONS OF CHAGRES RIVER
AT BOHIO GAUGING STATION
Areas Computed to Elev. +3 above M.S.L.
Date Area
Feb. 27, 1909 650 Sq. Ft.
April 30, 1909 750 " "
May 24, 1909 825 " "
July 13, 1909 710 " "
Dec. 23, 1909 625 " "
Jan. 11, 1910 590 " "
Feb. 27, 1909
April 30, 1909
July 13, 1909
May 24, 1909
Dec. 23, 1909
Jan. 11, 1910
Distance, in Feet, from Initial Point of Soundings.

Fig. 23.

of the annual minimum and maximum discharge at Alhajuela is about 3.25, at Gamboa 3.40, at Bohio 2.56, and at Gatun 2.50, decrease down the river being due to the regulation of the flood discharge by flowage and storage in the swamps and low areas. All the diagrams show the influence of the heavy rainfall during November and December, 1909. Figs. 30 and 31 show the conditions of discharge duration for the period used for Alhajuela and Gatun, together with the mean discharge of the several months placed in order of dryness. Comparing the diagrams, it is seen that the discharge of the three dry months at Alhajuela is about one-half that of the whole drainage basin, while the average discharge at Gatun is about three times that at Alhajuela, which is about the same as the ratio of the respective drainage areas. These conditions of minimum and average discharge, when considered with relation to the character of the country in the upper and lower portions of the drainage basin, offer additional evidence of the harmful effect of swamps, as far as conservation of water is concerned, although their value as flood regulators has been indicated.

TABLE 33.—Run-off Past Gatun, Arranged in Periods of Maximum Flow.

Period.	Date of Period (inclusive).	Discharge, in cubic feet per second.
1 Month	December, 1893	30 037
2 Months.......	November 1st, to December 31st, 1909........	27 135
3 "	October 1st, " December 31st, 1909........	22 287
4 "	October 1st, 1909, " January 31st, 1910........	19 650
5 "	August 1st, " December 31st, 1893........	18 470
6 "	July 1st. " December 31st, 1893........	17 460
7 "	July 1st, 1893, " January 31st, 1894........	16 600
8 "	June 1st, 1893, " January 31st, 1894........	15 520
9 "	May 1st, 1893, " January 31st, 1894........	14 650
10 "	April 1st, 1893, " January 31st, 1894........	13 740
11 "	April 1st, 1893, " February 28th, 1894........	12 743
1 Year.........	February 1st, 1893, " January 31st, 1894........	11 948
2 Years.........	March 1st, 1892, " February 28th, 1894........	11 366
3 "	March 1st, 1892, " February 28th, 1895........	10 736
4 "	November 1st, 1891, " October 31st, 1895........	10 003
5 "	February 1st, 1890, " January 31st, 1895........	9 890
6 "	March 1st, 1890, " February 29th, 1896........	9 460
7 "	January 1st, 1890, " December 31st, 1896........	9 070
8 "	May 1st, 1890, " April 30th, 1898........	8 951
9 "	January 1st, 1890. " December 31st, 1898........	8 632
10 "	January 1st, 1890, " December 31st, 1899........	8 318
11 "	January 1st, 1890, " December 31st, 1900........	8 080
12 "	May 1st, 1890, " April 30th, 1902........	8 020
13 "	January 1st, 1890, " December 31st, 1902........	7 840
14 "	May 1st, 1890, " April 30th, 1904........	7 780
15 "	January 1st, 1890, " December 31st, 1904........	7 680
16 "	January 1st, 1890, " December 31st, 1905	7 485
17 "	January 1st, 1890, " December 31st, 1906........	7 430
18 "	January 1st, 1890, " December 31st, 1907........	7 340
19 "	January 1st, 1892, " December 31st, 1910........	7 481
20 "	January 1st, 1891, " December 31st, 1910........	7 472
21 "	January 1st, 1890, " December 31st, 1910........	7 602

PLATE II.
PAPERS, AM. SOC. C. E.
JANUARY, 1913.
SAVILLE ON
HYDROLOGY OF THE
PANAMA CANAL.

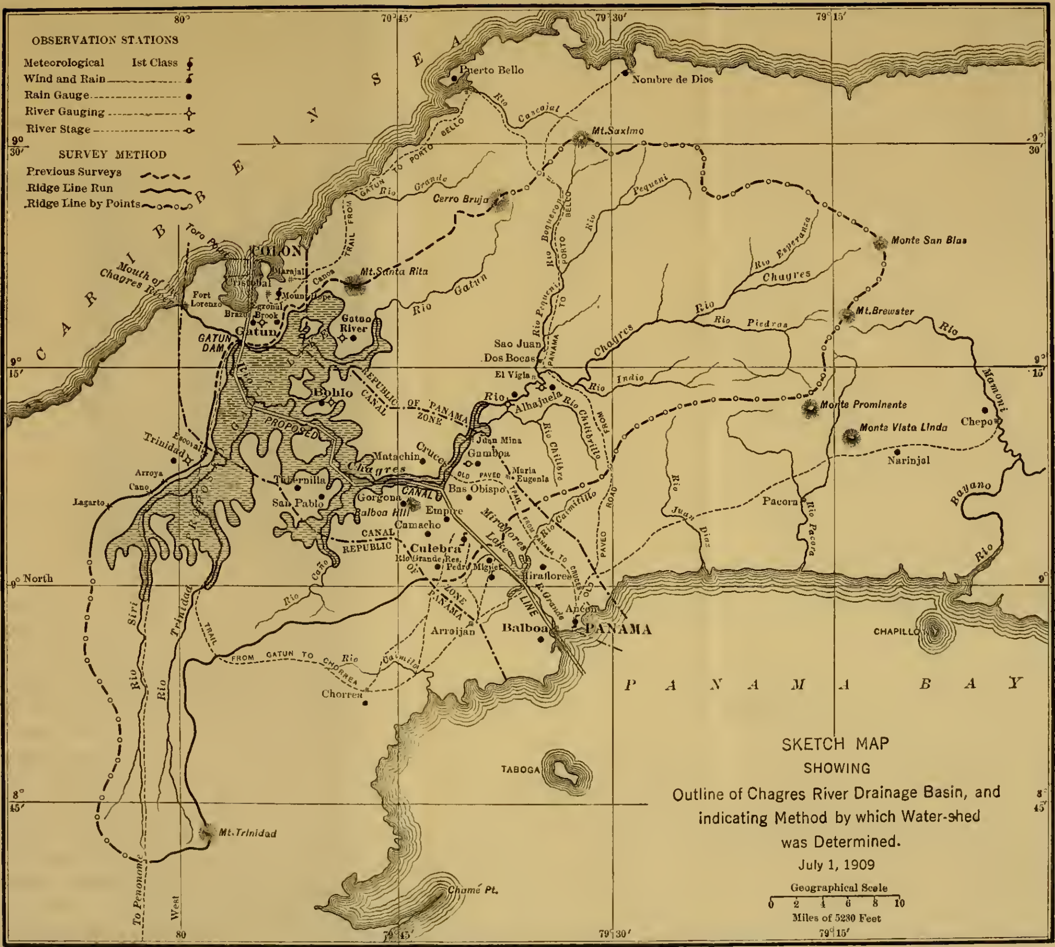


TABLE 34.—Run-off Past Gatun, Arranged in Periods of Minimum Flow.

Period.	Date of Period (inclusive).	Discharge, in cubic feet per second.
1 Month	March, 1908	851
2 Months	March 1st, to April 30th, 1908	872
3 "	February 1st, " April 30th, 1908	930
4 "	January 1st, " April 30th, 1908	1 191
5 "	January 1st, " May 31st, 1903	1 739
6 "	January 1st, " June 30th, 1903	2 136
7 "	December 1st, 1902, " June 30th, 1903	2 526
8 "	December 1st, 1902, " July 31st, 1903	3 025
9 "	January 1st, 1905, " September 30th, 1905	3 486
10 "	December 1st, 1904, " September 30th, 1905	3 814
11 "	July 1st, 1905, " May 31st, 1906	4 449
1 Year	June 1st, 1905, " May 31st, 1906	4 435
2 Years	September 1st, 1899, " August 31st, 1901	5 163
3 "	September 1st, 1898, " August 31st, 1901	5 310
4 "	August 1st, 1904, " July 31st, 1908	5 544
5 "	August 1st, 1898, " July 31st, 1903	5 815
6 "	July 1st, 1902, " June 30th, 1908	5 823
7 "	July 1st, 1899, " June 30th, 1906	5 928
8 "	August 1st, 1898, " July 31st, 1906	5 902
9 "	May 1st, 1899, " April 30th, 1908	5 972
10 "	August 1st, 1898, " July 31st, 1908	5 937
11 "	February 1st, 1898, " January 31st, 1909	6 032
12 "	July 1st, 1896, " June 30th, 1908	6 255
13 "	November 1st, 1895, " October 31st, 1908	6 239
14 "	February 1st, 1895, " January 31st, 1909	6 314
15 "	February 1st, 1894, " January 31st, 1909	6 749
16 "	November 1st, 1893, " October 31st, 1909	6 769
17 "	November 1st, 1892, " October 31st, 1909	6 970
18 "	January 1st, 1891, " December 31st, 1908	7 092
19 "	November 1st, 1890, " October 31st, 1909	7 138
20 "	November 1st, 1890, " October 31st, 1910	7 411
21 "	January 1st, 1890, " December 31st, 1910	7 602

The critical period in maximum discharge appears to be a space of about 1 month in the case of Gatun; that is, a fairly uniformly increasing discharge can be expected for about 11 months of the year, with the rate in the twelfth month greatly in excess.

At Alhajuela this period of excess flow is of shorter duration, being only about 10 months out of the 252 recorded, or an average of about 15 days per year.

Fig. 32 shows the interrelations of discharge at the four main stations on the Chagres River.

Figs. 33, 34, and 35 show the cycle of monthly average discharge, or the average hydrograph, for the years of record. From these and the preceding diagrams the probable quantity of water available for power is easily seen. For the design of spillway and discharge channels, absolute maximum conditions are required, and these may be found in Table 35.

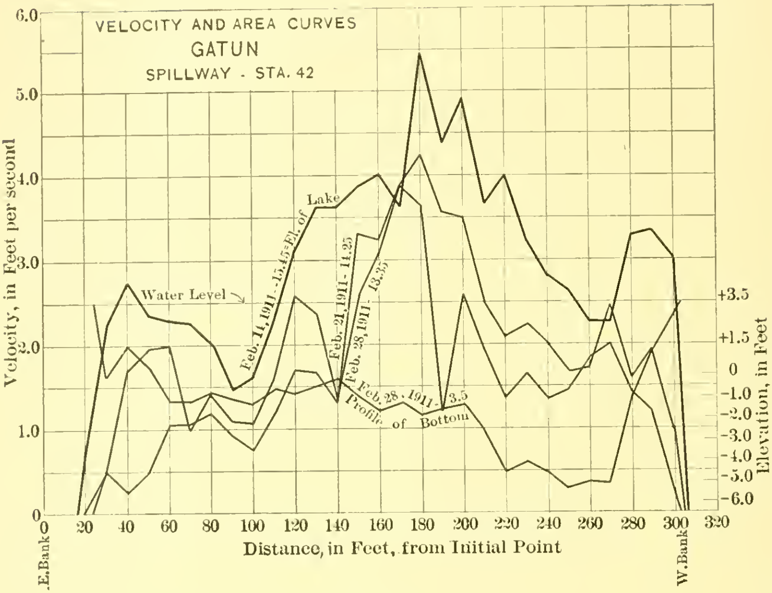

VELOCITY AND AREA CURVES
GATUN
SPILLWAY - STA. 42
Velocity, in Feet per second
Elevation, in Feet
Distance, in Feet, from Initial Point
E.Bank
W.Bank
Water Level
Feb. 11, 1911 - 15.45=El. of Lake
Feb. 21, 1911 - 14.25
Feb. 28, 1911 - 13.35
Feb. 28, 1911 - 3.5
Profile of Bottom
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320
0 1.0 2.0 3.0 4.0 5.0 6.0
+3.5 +1.5 0 -1.0 -2.0 -3.0 -4.0 -5.0 -6.0

FIG. 24.

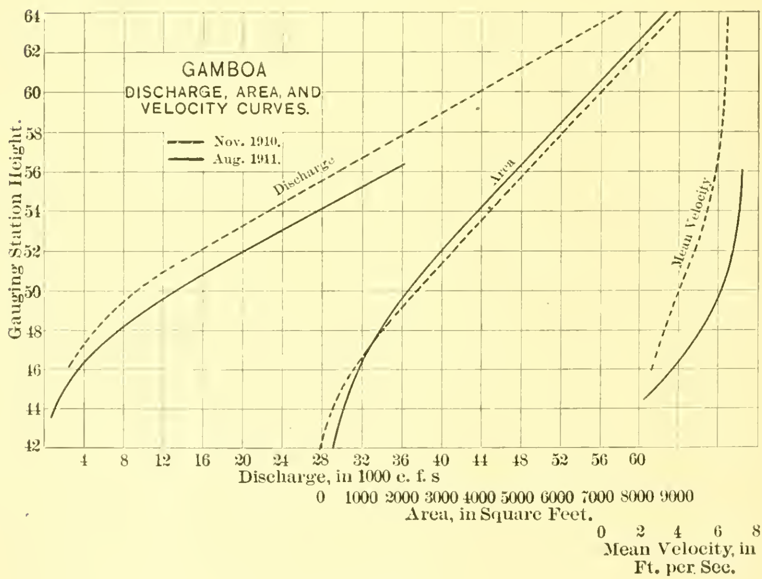

GAMBOA
DISCHARGE, AREA, AND
VELOCITY CURVES.
Nov. 1910.
Aug. 1911.
Discharge
Area
Mean Velocity
Gauging Station Height.
42 44 46 48 50 52 54 56 58 60 62 64
4 8 12 16 20 24 28 32 36 40 44 48 52 56 60
Discharge, in 1000 c. f. s
0 1000 2000 3000 4000 5000 6000 7000 8000 9000
Area, in Square Feet.
0 2 4 6 8
Mean Velocity, in
Ft. per Sec.

FIG. 25.

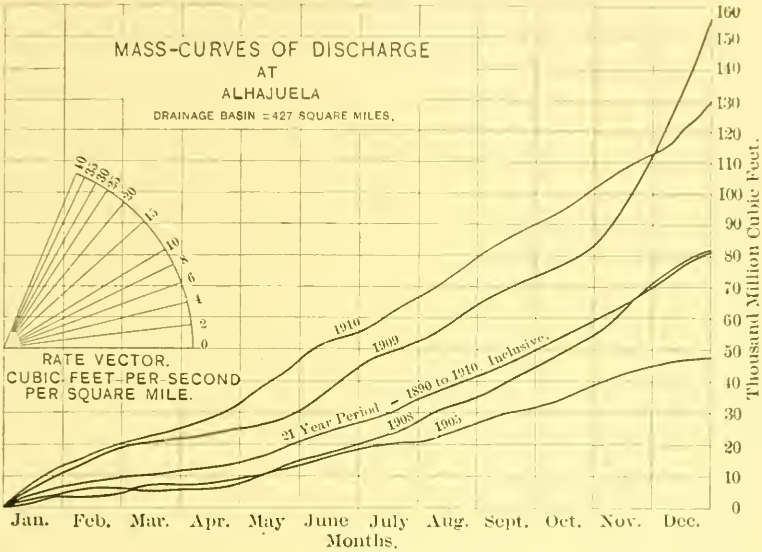

MASS-CURVES OF DISCHARGE
AT
ALHAJUELA
DRAINAGE BASIN = 427 SQUARE MILES.
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE.
1910
1909
21 Year Period - 1890 to 1910, Inclusive.
1908
1905
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
Months.
Thousand Million Cubic Feet.

Fig. 26.

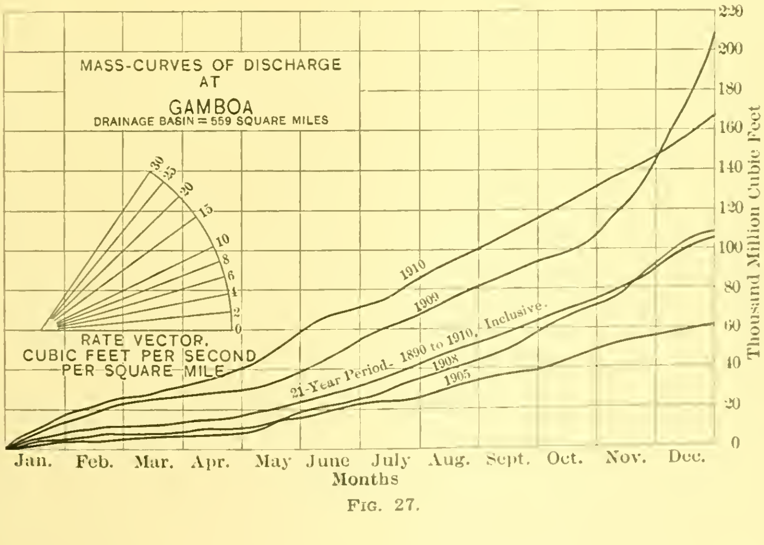

MASS-CURVES OF DISCHARGE
AT
GAMBOA
DRAINAGE BASIN = 559 SQUARE MILES
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE
1910
1909
21-Year Period- 1890 to 1910, Inclusive.
1908
1905
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
Months
Thousand Million Cubic Feet

Fig. 27.

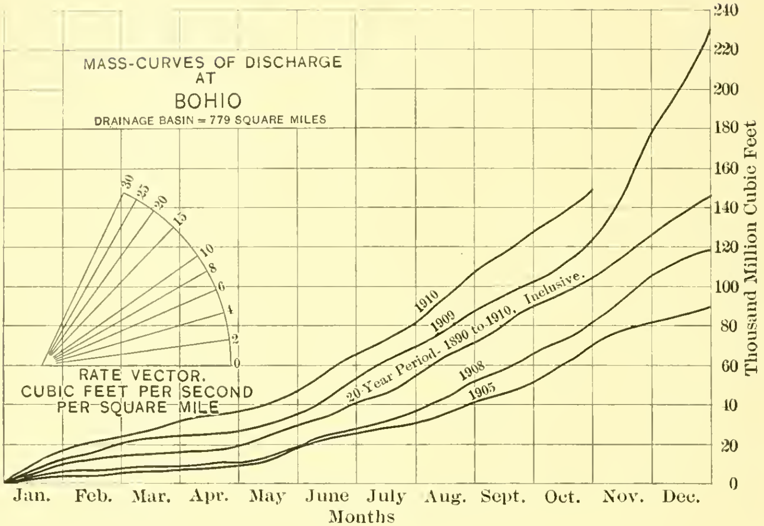

MASS-CURVES OF DISCHARGE
AT
BOHIO
DRAINAGE BASIN = 779 SQUARE MILES
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE
30
25
20
15
10
8
6
4
2
0
1910
1909
20-Year Period- 1890 to 1910, Inclusive.
1908
1905
240
220
200
180
160
140
120
100
80
60
40
20
0
Thousand Million Cubic Feet
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Months

FIG. 28.

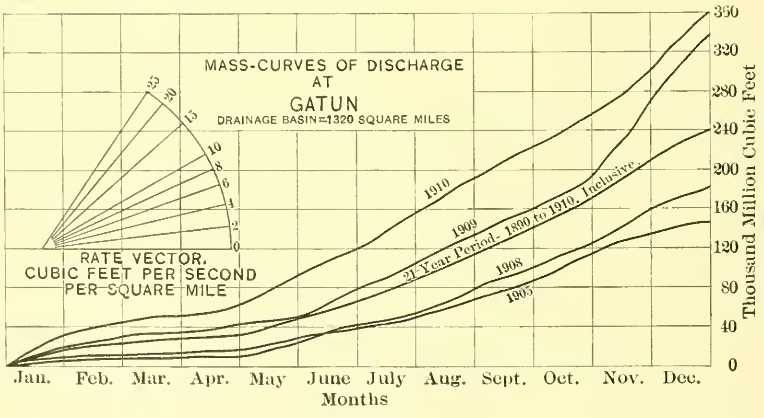

MASS-CURVES OF DISCHARGE
AT
GATUN
DRAINAGE BASIN=1320 SQUARE MILES
RATE VECTOR.
CUBIC FEET PER SECOND
PER SQUARE MILE
25
20
15
10
8
6
4
2
0
1910
1909
21-Year Period- 1890 to 1910, Inclusive.
1908
1905
360
320
280
240
200
160
120
80
40
0
Thousand Million Cubic Feet
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Months

FIG. 29.

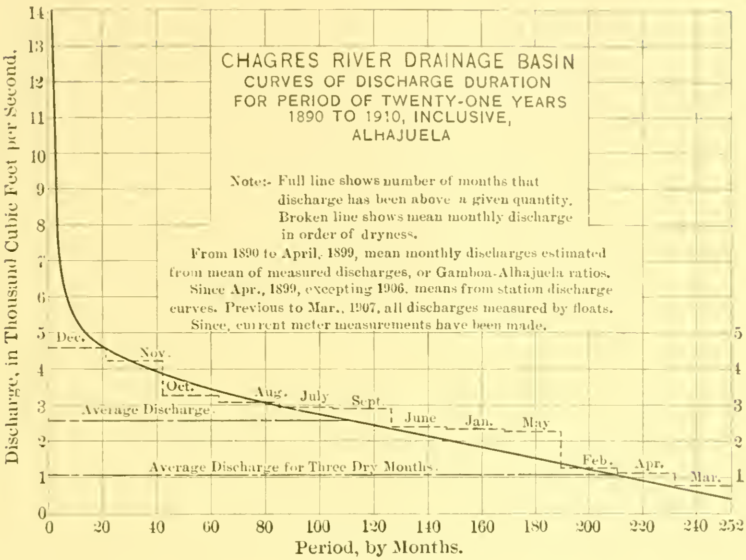

CHAGRES RIVER DRAINAGE BASIN
CURVES OF DISCHARGE DURATION
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE,
ALHAJUELA
Note:- Full line shows number of months that discharge has been above a given quantity. Broken line shows mean monthly discharge in order of dryness.
From 1890 to April, 1899, mean monthly discharges estimated from mean of measured discharges, or Gamboa-Alhajuela ratios. Since Apr., 1899, excepting 1906, means from station discharge curves. Previous to Mar., 1907, all discharges measured by floats. Since, current meter measurements have been made.
Discharge, in Thousand Cubic Feet per Second.
Period, by Months.
Average Discharge.
Average Discharge for Three Dry Months.
Dec.
Nov.
Oct.
Aug.
July
Sept.
June
Jan.
May
Feb.
Apr.
Mar.

FIG. 30.

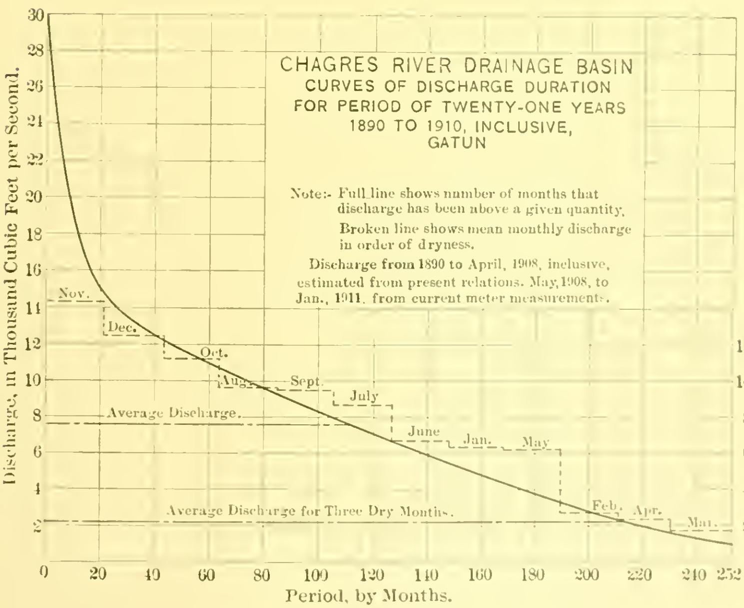

CHAGRES RIVER DRAINAGE BASIN
CURVES OF DISCHARGE DURATION
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE,
GATUN
Note:- Full line shows number of months that discharge has been above a given quantity. Broken line shows mean monthly discharge in order of dryness.
Discharge from 1890 to April, 1908, inclusive, estimated from present relations. May, 1908, to Jan., 1911, from current meter measurements.
Discharge, in Thousand Cubic Feet per Second.
Period, by Months.
Average Discharge.
Average Discharge for Three Dry Months.
Nov.
Dec.
Oct.
Aug.
Sept.
July
June
Jan.
May
Feb.
Apr.
Mar.

FIG. 31.

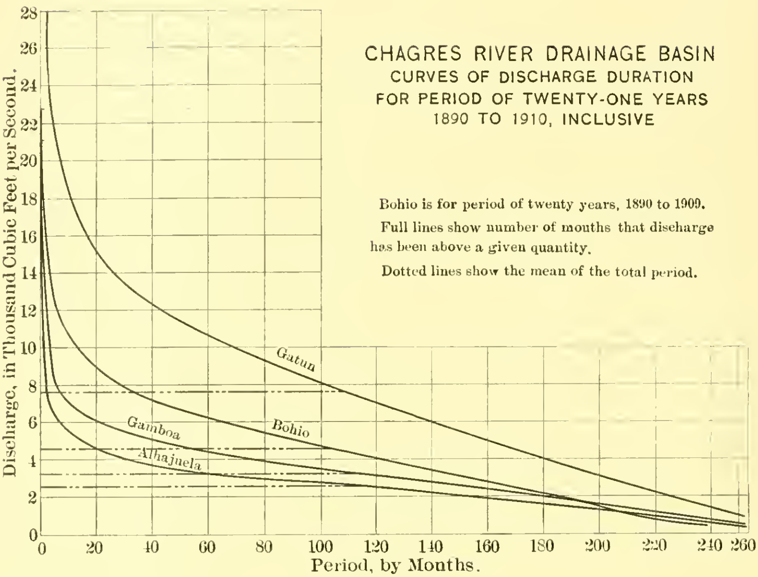

CHAGRES RIVER DRAINAGE BASIN
CURVES OF DISCHARGE DURATION
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE
Bohio is for period of twenty years, 1890 to 1909.
Full lines show number of months that discharge has been above a given quantity.
Dotted lines show the mean of the total period.
Discharge, in Thousand Cubic Feet per Second.
Period, by Months.
Gatun
Bohio
Gamboa
Alhajuela
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
0
20
40
60
80
100
120
140
160
180
200
220
240
260

FIG. 32.

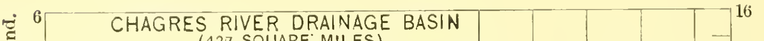

CHAGRES RIVER DRAINAGE BASIN
(427 SQUARE MILES)
CYCLE OF AVERAGE MONTHLY DISCHARGE
FOR PERIOD OF TWENTY-ONE YEARS
1890 TO 1910, INCLUSIVE.
ALHAJUELA
Note:- From 1890 to April, 1899, mean monthly discharges estimated from mean of measured discharges, or Gamboa-Alhajuela ratios. Since Apr., 1899, excepting 1906, means from station discharge curves. Previous to Mar., 1907, all discharges were measured by floats. Since that date current meter measurements have been made.
Annual Average Discharge.
Average Discharge for Three Dry Months.
Discharge, in Thousand Cubic Feet per Second.
Inches per Month on Water-shed.
Months.
Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar.
0
1
2
3
4
5
6
0
2
4
6
8
10
12
14
16

FIG. 33.

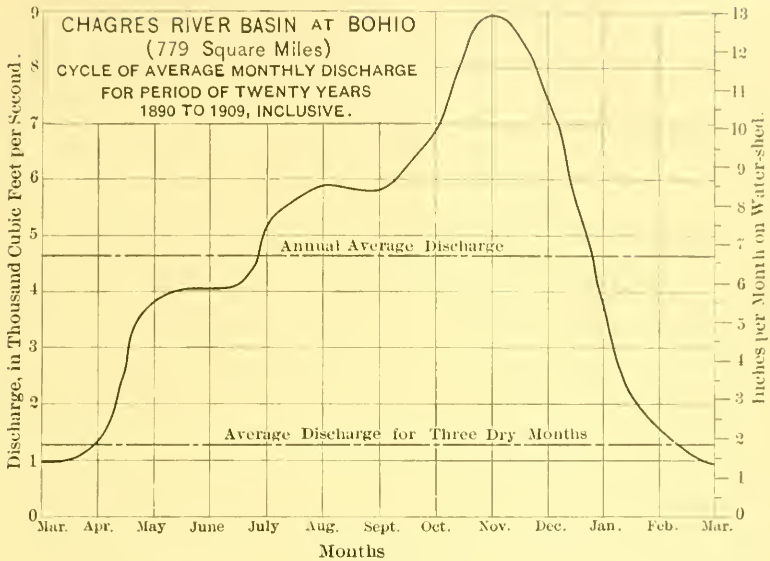

CHAGRES RIVER BASIN AT BOHIO
(779 Square Miles)
CYCLE OF AVERAGE MONTHLY DISCHARGE
FOR PERIOD OF TWENTY YEARS
1890 TO 1909, INCLUSIVE.
Annual Average Discharge
Average Discharge for Three Dry Months
Discharge, in Thousand Cubic Feet per Second.
Inches per Month on Water-shed.
Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar.
Months

FIG. 34.

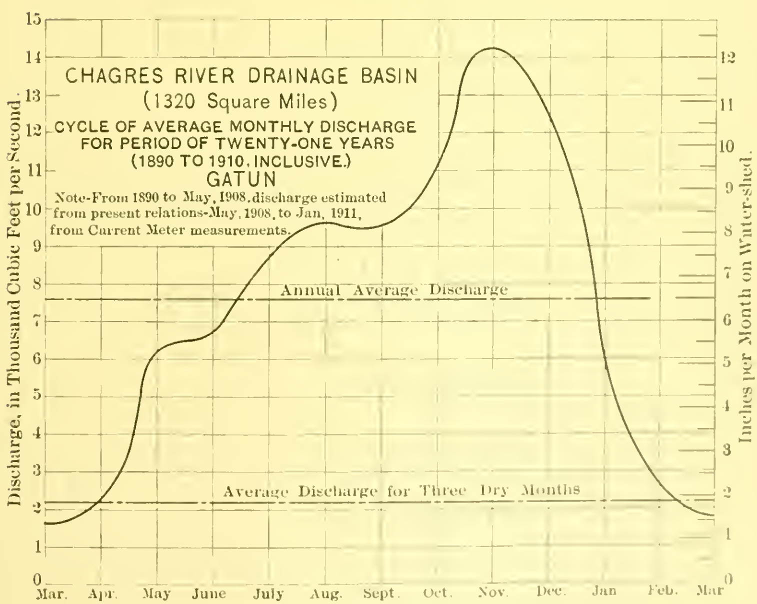

CHAGRES RIVER DRAINAGE BASIN
(1320 Square Miles)
CYCLE OF AVERAGE MONTHLY DISCHARGE
FOR PERIOD OF TWENTY-ONE YEARS
(1890 TO 1910, INCLUSIVE.)
GATUN
Note-From 1890 to May, 1908, discharge estimated from present relations-May, 1908, to Jan, 1911, from Current Meter measurements.
Annual Average Discharge
Average Discharge for Three Dry Months
Discharge, in Thousand Cubic Feet per Second.
Inches per Month on Water-shed.
Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan Feb. Mar
Months

FIG. 35.

TABLE 35.—MAXIMUM RATES OF RUN-OFF IN CHAGRES RIVER DRAINAGE BASIN, DURING FRESHETS BEGINNING DECEMBER 26TH, 1909, AND FEBRUARY 12TH, 1911.

Discharge = Cubic Feet per Second.

Height = Elevation above Mean Sea Level.

ALHAJUELA.—DRAINAGE AREA = 427 SQ. MILES.

Date.	Time.	Height.	Momentary rate of discharge.	24-Hour rate of discharge.
December 26th, 1909..	7 P. M.	121.00	170 000	95 700
December 30th, 1909..	11 A. M.	112.00	97 200	43 500
February 12th, 1911..	11 P. M.	105.50	43 600	17 280

BOHIO.—DRAINAGE AREA = 779 SQ. MILES.

Date.	Time.	Height.	Momentary rate of discharge.	24-Hour rate of discharge.
December 27th........	11 A. M.	38.7	90 000	78 400
December 31st........	2 A. M.	32.1	58 500	38 600
February 13th........	10 A. M.	21.1	37 500	......

GATUN.—DRAINAGE AREA = 1 320 SQ. MILES.

Date.	Time.	Height.	Momentary rate of run-off (storage + discharge).	24-Hour rate of run-off (storage + discharge).
December 28th........	9 A. M.	19.65	124 000	94 000
December 31st........	9 A. M.	17.7	60 000	57 000
February 13th........	4 P. M.	15.7	38 860	21 520

Floods are of frequent occurrence in the Chagres Basin, and from 1879 to 1906, General Abbot* records eight rises having a maximum discharge of more than 40 000 cu. ft. per sec. at Gamboa. Subsequent to this period, there have been other similar floods—those of November and December, 1909. That is, there have been eleven great floods in the Chagres in 30 years, or an average of about one every 3 years. The most important recent flood, and the one which gave the maximum recorded gauge height, was that of December 26th, 1909. This was accurately observed in all its conditions, and the data and results are believed to be the most reliable record of any flood on the Chagres River. The flood was the last of a series of three, all of which can be classed as great, and their occurrence between November 11th and December 31st, 1909, is an indication of the conditions for which it is necessary to provide in tropical water supply. During the period,

* "Problems of the Panama Canal," p. 155.

TABLE 36.—COMPARATIVE RAINFALL DURING FLOOD PERIODS.
December 26th, 27th, 28th, 29th, and 30th, 1909.
November 16th, 17th, 18th, 19th, and 20th, 1909.
December 1st, 2d, 3d, 4th, and 5th, 1906.

	1st Period.			2d Period.			3d Period.			4th Period.			5th Period.			Total. Dec., 1909, 26th to 30th, inclusive.	Total. Nov., 1909, 16th to 20th, inclusive.	Total. Dec., 1906, 1st to 5th, inclusive.
December, 1909.	26th.			27th.			28th.			29th.			30th.					
November, 1909.		16th.			17th.			18th.			19th.			20th.				
December, 1906.			1st.			2d.			3d.			4th.			5th.			
Nombre de Dios	3.43	0.81	......	2.38	7.08	......	1.79	6.64	......	0.57	4.49	......	3.15	0.48	......	11.32	19.50	....
Porto Bello	5.55	3.36	......	2.21	1.45	......	3.64	8.53	......	7.96	2.84	......	4.94	0.28	......	21.30	16.46	....
Cristobal	3.69	0.34	0.04	1.85	3.40	0.13	0.50	3.39	8.47	0.36	0.73	0.39	3.29	1.19	0.05	9.69	9.05	9.08
Brazos Brook	4.05	0.31	0.00	1.43	3.82	0.10	1.13	3.41	8.96	0.43	1.46	0.45	3.33	0.78	0.00	10.37	9.78	9.51
Gatun	3.99	1.19	0.02	3.84	3.64	0.25	0.21	3.74	10.48	0.30	1.63	0.41	2.36	0.56	0.06	10.70	10.76	11.25
Monte Lirio	5.35	0.66	......	1.00	3.83	......	0.62	3.18	......	0.10	1.55	......	2.77	0.45	......	9.84	9.67	....
Lagartera	3.10	0.60	......	0.51	4.51	......	0.61	2.98	......	0.18	1.42	......	2.35	0.43	......	6.75	9.84	....
Bobio	3.77	2.26	0.21	0.54	2.15	0.34	0.55	2.96	6.13	0.51	0.99	0.15	2.23	0.36	0.05	7.60	8.72	6.88
Empire	1.67	0.52	2.63	1.71	2.59	0.48	0.22	1.11	6.15	0.09	0.73	0.02	1.87	0.05	0.06	5.56	5.01	9.34
Camacho	1.32	1.10	2.21	2.11	3.20	0.15	0.41	1.10	5.53	0.07	1.46	0.10	2.61	0.08	0.00	6.52	6.94	7.99
Culebra	1.51	0.90	1.97	1.72	3.17	0.31	0.41	1.56	5.55	0.11	1.08	0.01	2.47	0.07	0.04	6.22	6.78	7.91
Gamboa	2.66	0.75	1.61	1.10	1.96	0.36	0.32	2.18	6.29	0.31	1.57	0.04	2.18	0.12	0.00	6.60	6.58	8.30
Alhajuela	5.56	0.31	0.66	1.70	3.76	0.28	3.49	3.62	8.19	1.09	2.22	0.14	2.91	0.54	0.00	14.75	10.45	9.27
Rio Grande	1.70	0.74	2.63	1.74	3.33	0.47	0.20	1.33	5.58	0.11	0.90	0.00	2.11	0.08	0.02	5.83	6.38	8.70
Balboa	2.71	0.19	0.50	1.28	1.67	0.21	0.01	1.75	2.40	0.25	0.32	0.20	1.30	0.06	0.02	5.55	3.99	3.33
Ancon	2.16	0.10	0.42	1.31	1.61	1.61	0.06	1.64	1.58	0.20	0.31	0.22	1.16	0.22	0.00	4.89	3.88	3.83

it is estimated that 128 000 000 000 cu. ft. of water were discharged past Gatun. This quantity, if delivered under the present conditions of Gatun Lake (water surface + 15) and with no outlet, would fill the lake to about Elevation + 73, or to within 14 ft. of maximum lake level (+ 87). Hydrographs of two of these floods and mass-curves of discharge are shown on Plates III and IV. These hydro-

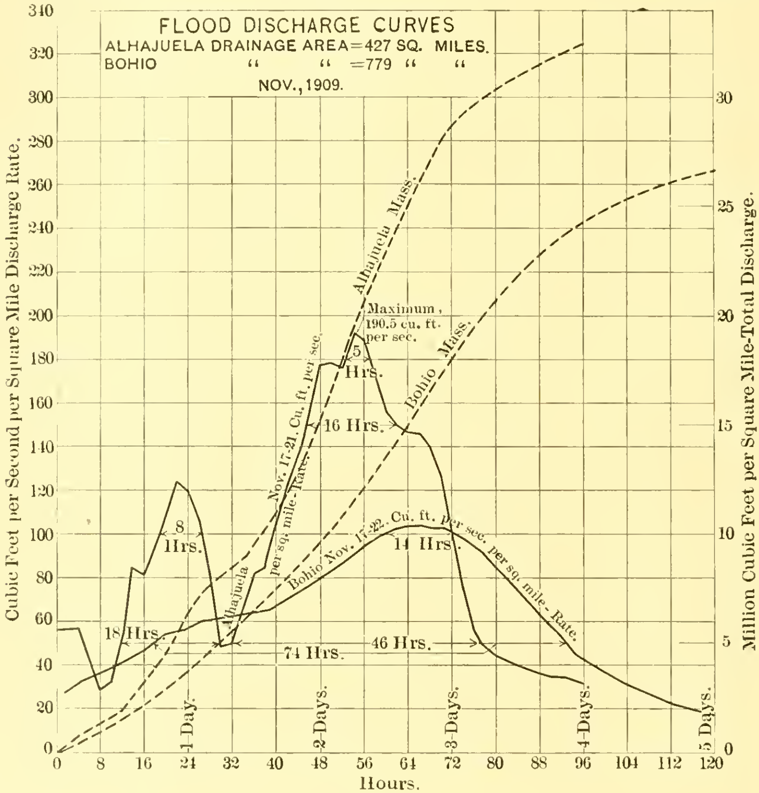


FIG. 36.

graphs are typical of the Chagres River freshets, and a study of them presents some very interesting facts. Figs. 36 and 37 show these conditions, and indicate two distinct types of freshet discharge which are probably of not infrequent occurrence in a long term of years, although they have been unusual during the period in which authentic records are available. The freshet of November, 1909, was of com-

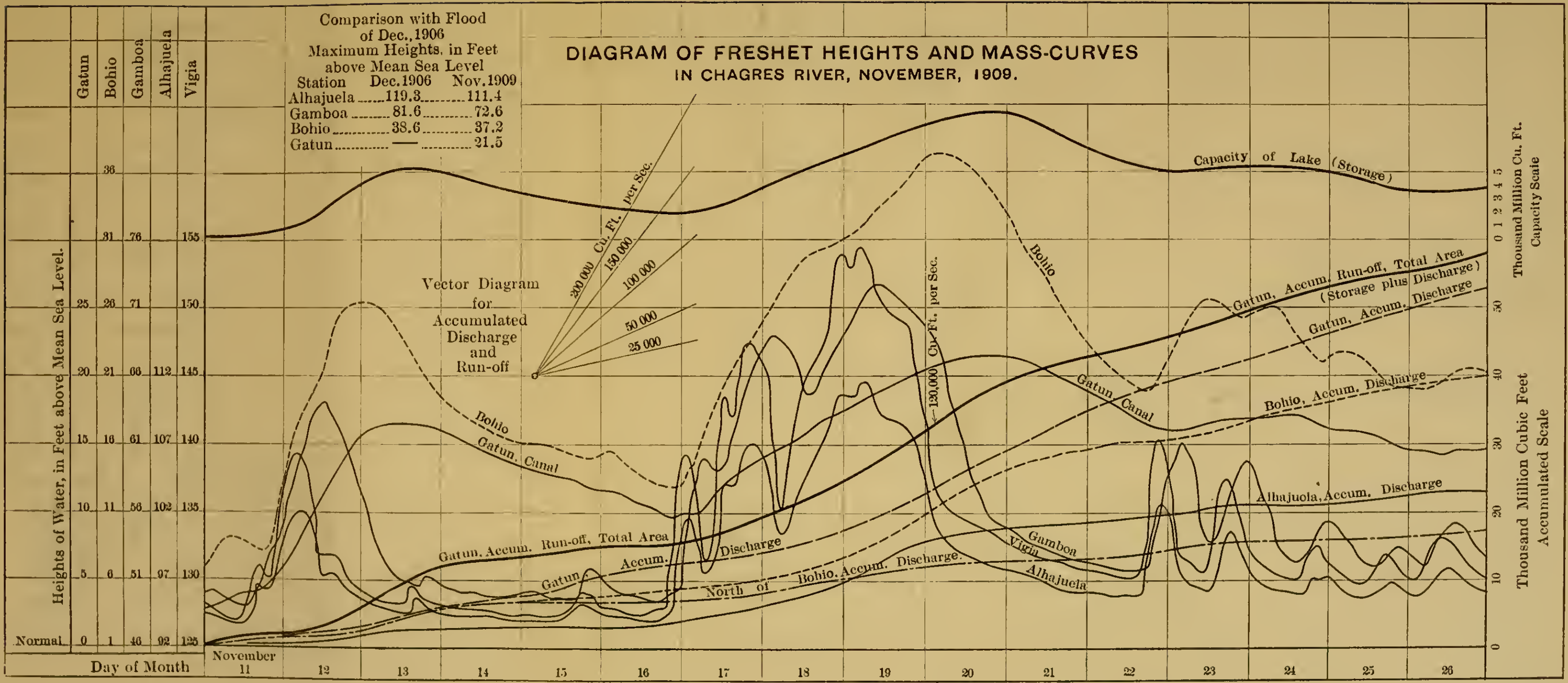


DIAGRAM OF FRESHET HEIGHTS AND MASS-CURVES
IN CHAGRES RIVER, NOVEMBER, 1909.
Comparison with Flood
of Dec., 1906
Maximum Heights, in Feet
above Mean Sea Level
Station Dec.1906 Nov.1909
Alhajuela 119.3 111.4
Gamboa 81.6 72.6
Bohio 38.6 37.2
Gatun — 21.5
Vector Diagram
for
Accumulated
Discharge
and
Run-off
200 000 Cu. Ft. per Sec.
150 000
100 000
50 000
25 000
120,000 Cu. Ft. per Sec.
Capacity of Lake (Storage)
Gatun, Accum. Run-off, Total Area
(Storage plus Discharge)
Gatun, Accum. Discharge
Bohio, Accum. Discharge
Alhajuela, Accum. Discharge
Gatun, Accum. Discharge
North of Bohio, Accum. Discharge
Bohio
Gatun, Canal
Gamboa
Vigia
Alhajuela
Heights of Water, in Feet above Mean Sea Level.
Gatun
Bohio
Gamboa
Alhajuela
Vigia
Normal
Day of Month
November
Thousand Million Cu. Ft.
Capacity Scale
Thousand Million Cubic Feet
Accumulated Scale

paratively longer duration and of more widely distributed rainfall than that of the succeeding month, although the crest height at all stations was less than that of the December freshet. For the November rise, however, the discharge past Gatun was nearly 50% more than that of

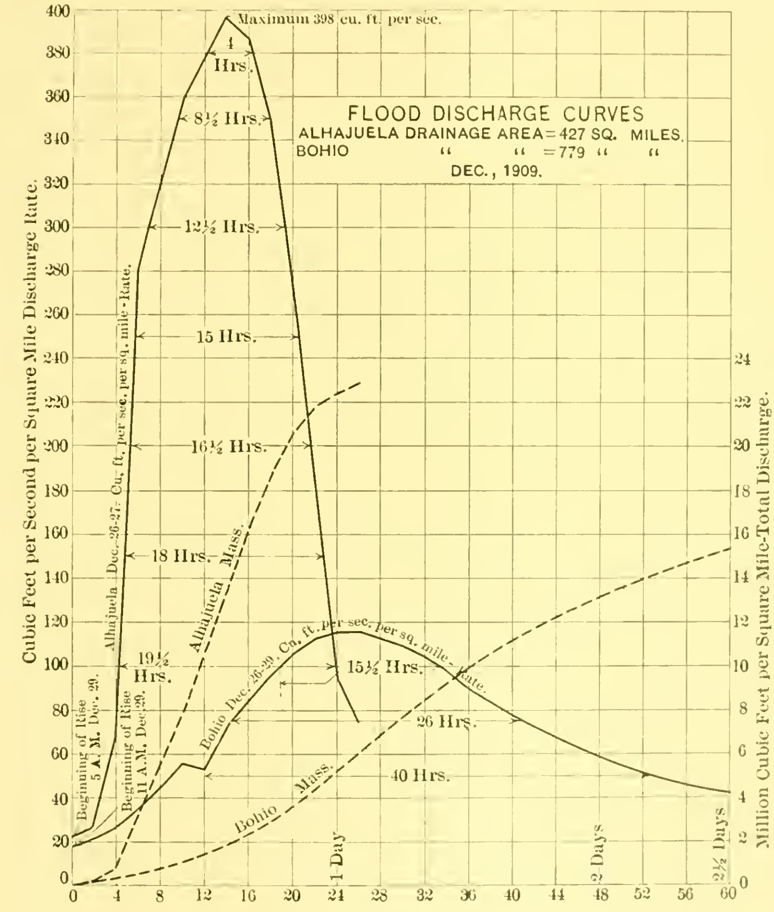


FIG. 37.

December. The duration of those extensive flows is a matter of considerable interest, and the length of time the rate of run-off equalled or exceeded a given rate is shown by the figures on the horizontal lines included between the discharge lines.

On comparing these two curves on this basis, it is seen that in November the flow was more than 100 cu. ft. per sec. per sq. mile for 40 hours, while in the case of the December freshet this rate was equalled or exceeded for only 19½ hours. During this time, however, the range above the 100 was three times that of the previous month. It appears, therefore, that, in caring for freshet flows in these localities, provision must be made for a rate of run-off per square mile of at least 300 cu. ft. per sec. per sq. mile continuing for 20 hours, and that total discharges of 32 000 000 cu. ft. per sq. mile for water-sheds having an area of 400 sq. miles, and 26 000 000 cu. ft. per sq. mile for those of twice this area, delivered in from 4 to 5 days, are not uncommon. In the Northern States, if provision is made for a run-off of 6 in. per 24 hours, it is thought to be conservative. The freshet of December, 1909, delivered at about twice this rate. The curves, of course, are only a different way of presenting the data already shown in Plate III and Fig. 36 (Chagres River freshet).

The Chagres River freshets are usually not a single rise and fall of the river, but a series of rises following each other more or less closely, depending on the rainfall and the portion of the drainage area in which the storm begins or is centered. The smaller rises which appear are often due principally to the fact that the larger rises have so closely preceded, and in many cases the former would not have been so noticeable except for the fact that the ground had just been so thoroughly soaked that most of the rainfall found its way very quickly into the stream. Table 37 shows the total run-off at Alhajuela, Bohio, and Gatun for the freshet of December 26th, 1909, and another of February 12th, 1910. This latter freshet is principally remarkable for occurring during the period when rainfall and stream flow are usually at the minimum condition, while the former freshet occurred during the period of maximum rainfall and river flow. The columns under Item (1) show the actual discharge conditions, while those under Item (2) give the estimated discharge due to the freshet in addition to the normal river flow, estimating the latter from the gauge height just before and after the rise. In the absence of rain gauges on the area above Alhajuela, the rainfall seems to be best estimated by taking the average of the Porto Bello and Alhajuela records. In the freshet of December 26th, this rainfall was about 19.50 in., and the discharge due to the particular conditions at this time was about

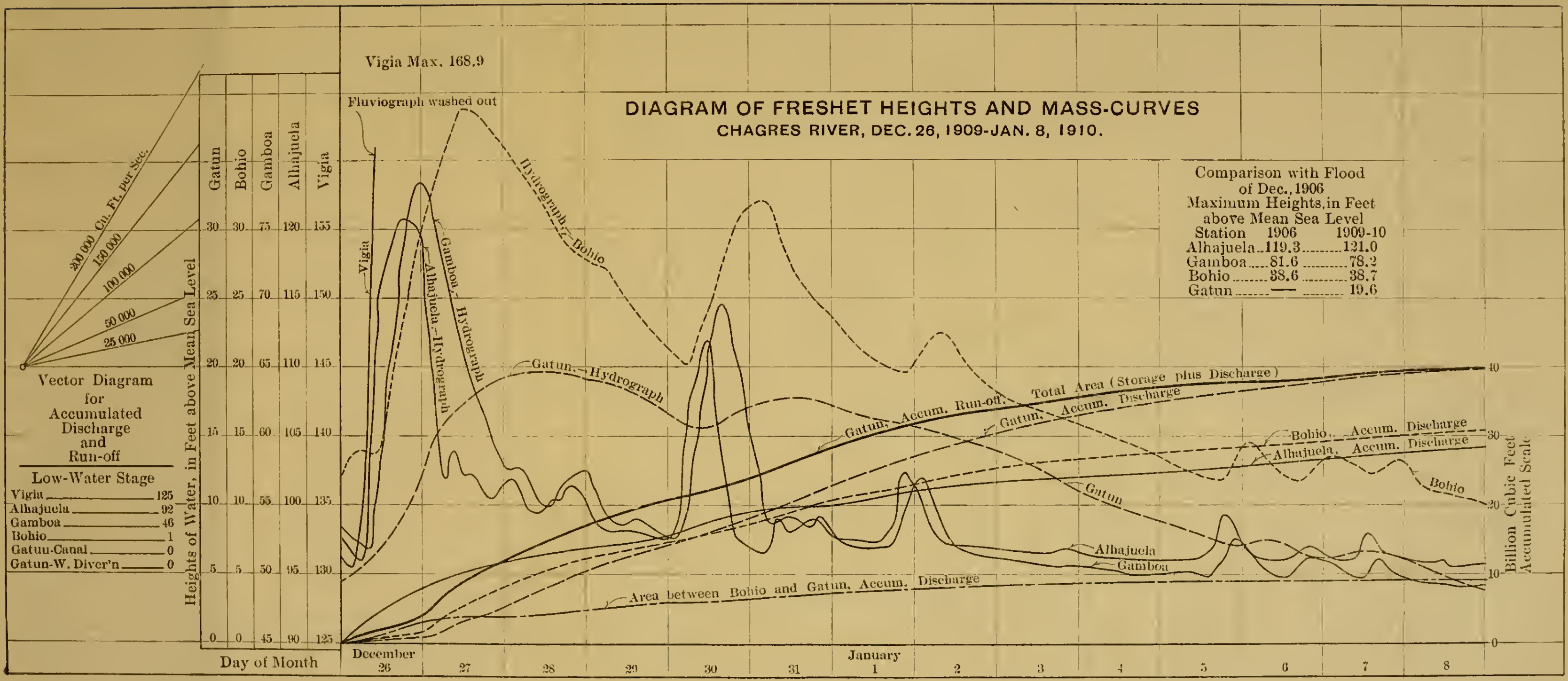


DIAGRAM OF FRESHET HEIGHTS AND MASS-CURVES
CHAGRES RIVER, DEC. 26, 1909-JAN. 8, 1910.

17.18 in., showing that nearly this entire rainfall was discharged during the period of 4 or 5 days succeeding. An inspection of Table 36 will show that the rise of December 26th-27th was due to rainfall which was quite general over the basin, while that of the 20th was due almost entirely to rainfall in the extreme eastern part of the territory, in the neighborhood of Porto Bello. This latter condition was also true in the freshet of February, 1910, when 4.38 in. of rain fell at Porto Bello, 1.62 in. at Alhajuela, and less than ¼ in. elsewhere in the whole basin.

TABLE 37.—CHAGRES RIVER DRAINAGE BASIN.
FRESHETS BEGINNING DECEMBER 26TH, 1909, AND FEBRUARY 12TH, 1911.
Accumulated Discharges.

DATE.	ALHAJUELA. Drainage area = 427 sq. miles.			BOHIO. Drainage area = 779 sq. miles			GATUN. Drainage area = 1 320 sq. miles.		
	Thousand million cubic feet.	Million cubic feet per square mile.	Total inches of run-off.	Thousand million cubic feet.	Million cubic feet per square mile.	Total inches of run-off.	Thousand million cubic feet.	Million cubic feet per square mile.	Total inches of run-off.
ITEM (1) INCLUDING THE RIVER FLOW AT TIME FRESHET BEGAN.									
Dec. 26th-Jan. 8th 14 days.	28.40	66.5	28.56	31.0	39.8	17.22	40.0	30.3	13.02
Feb. 12th-Feb. 15th 3.5 days.	2.35	5.5	2.35	....	....	....	3.02	2.28	0.98
ITEM (2) NOT INCLUDING RIVER FLOW—ACTUAL RUN-OFF DUE TO FRESHET.									
Dec. 26th-Jan. 8th 14 days.	17.27	40.45	17.18	18.66	23.96	10.30	25.06	19.00	8.18
Feb. 12th-Feb. 15th 3.5 days.	1.92	4.51	1.94	....	....	....	2.41	1.83	0.78

The freshet of December 26th gave the greatest momentary discharge at all gauging stations, while that of November gave the greatest quantity of water. The comparative relations of the former are given in Table 38.

TABLE 38.

	Maximum Discharge for Short Period.		Ratio to Mean Discharge.		
	Cubic feet per second.	Cubic feet per second per square mile.	December.	Annual.	Minimum month.
Albajuela.............	170 000	398	37.2	66.0	222
Gatun..................	124 000	94	10.0	16.4	76

Fig. 38 shows the hydrograph and discharge curves for the "dry season" freshet of February, 1910, and it is interesting to note that the total run-off at Gatun was about one-fourth of the discharge of the "wet season" freshet of December, 1909. Figs. 39 and 40 show the surface slope of the Chagres River during freshet periods, with the crest at various places. The time the crest passed the gauging stations

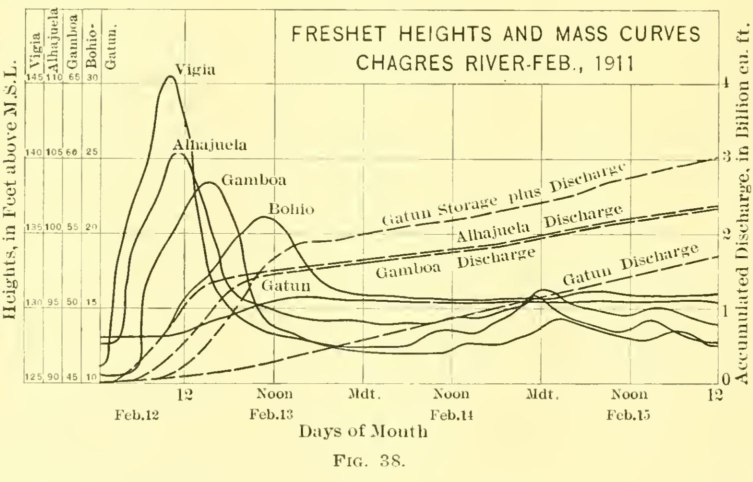
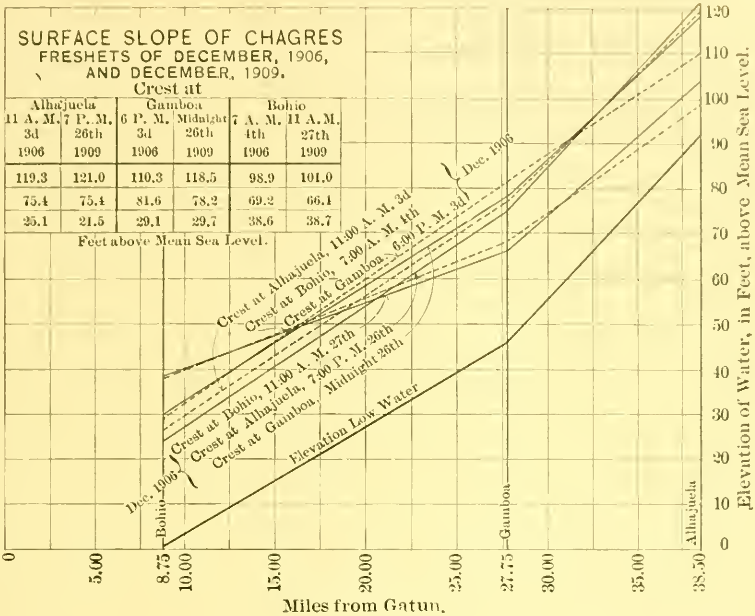


FIG. 38.

is given on Fig. 39, and from this it is possible to note the progress of the crest down the river. Table 39, for the freshet of December 26th, 1909, shows these data in concrete form.

The Gamboa station had not been established in December, 1909, but the flow past the station at Alhajuela had a mean maximum velocity of 13.57 ft. per sec., or more than four times that with which the crest moved down stream. In the lesser rise of February, 1911, the



Alhajuela 11 A. M. 3d 1906	Alhajuela 7 P. M. 26th 1909	Gamboa 6 P. M. 3d 1906	Gamboa Midnight 26th 1909	Bohio 7 A. M. 4th 1906	Bohio 11 A. M. 27th 1909
119.3	121.0	110.3	118.5	98.9	101.0
75.4	75.4	81.6	78.2	69.2	66.4
25.1	21.5	29.1	29.7	38.6	38.7

Fig. 39.

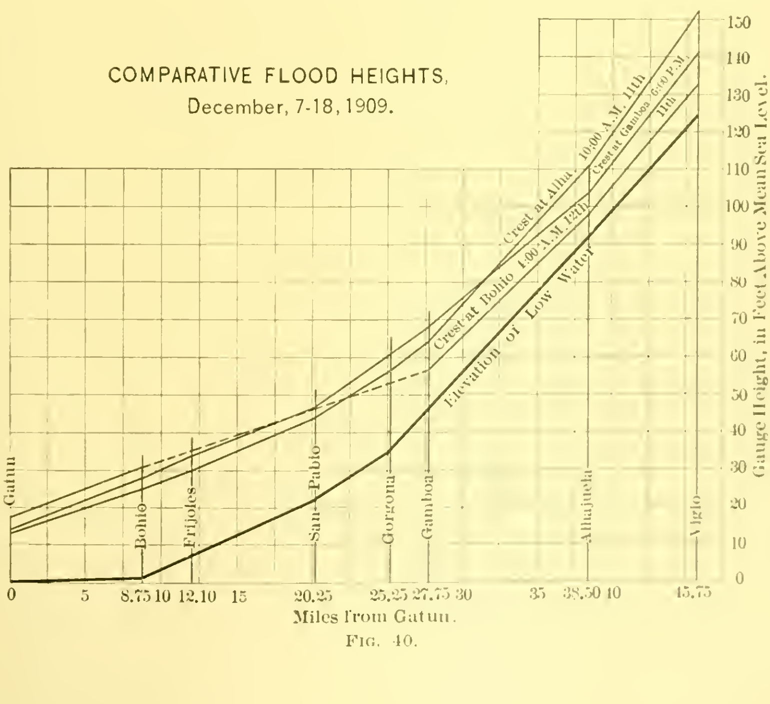


Fig. 40.

mean maximum velocity at Alhajuela was 7.85 ft. per sec., and at Gamboa 6.66 ft. per sec., while the crest moved between these stations at a rate of 2.35 miles per hour, or 3.45 ft. per sec.

TABLE 39.

Station.	Distance from preceding station, in miles.	Time of crest.	Time to next station, in hours.	Mean Velocity.		Surface Slope, in feet per mile. Crest at:	
				Miles per hour.	Feet per second.	Alhajuela.	Gamboa
Alhajuela	0.0	7 P. M. 26th	0.0	0.0	0.0	0.0	0.0
Gamboa	10.75	Midnight 26th	5.0	2.14	3.14	4.25	3.75
Bohio	19.00	11 A. M. 27th	11.0	1.73	2.54	2.68	2.56

The foregoing facts are of especial interest in the study of river hydraulics, and appear to indicate: (1) That flood crests move down stream with about the same velocity, irrespective of gauge height, due to the configuration of the average cross-section and the meanderings of the stream. (2) That the actual mean stream velocity is absolutely dependent on the gauge height. In the high rises in the Chagres River, the maximum current velocity at any point may be more than four times that of the crest, and gradually decreases in rate with the gauge height until, on the low flows, the two velocities practically coincide.

Run-off or stream flow depends, not only on the quantity of precipitation, but also on its distribution throughout the period under consideration. In the case of the Chagres River freshets, it has been shown that after a period of heavy rainfall the ground has become so thoroughly saturated that nearly all of a subsequent storm may pass quickly to the stream. On the other hand, after a period of drought and dry weather, oftentimes very little of the rainfall finds its way to the stream, or else it comes so slowly as to be unnoticed. While excessive downpours of short duration may be disastrous to small reservoirs in some cases, as a general proposition, they are so local in their happenings as to be of interest mainly in combination with the design of sewers and drains. The effect of such conditions, therefore, is best studied on small areas. The Rio Grande Reservoir and water-shed have offered opportunities for such investigation. This reservoir is about 1 mile south of Culebra, at the extreme head-waters

of the Rio Grande. The top of the spillway is at Elevation 235.2, and the surface of the lake can be raised 3 ft. by the use of flash-boards. At the latter elevation the area of the reservoir surface is about 0.09 sq. mile, and the total area of the basin above the dam and including the reservoir is 3.15 sq. miles. The land area is wholly uninhabited, is covered with tropical vegetation and second growth, and is representative of the ordinary woodland conditions which obtain in the greater part of the river basins on the Isthmus.

TABLE 40.—EXCESSIVE RAINFALL AND RUN-OFF IN THE RIO GRANDE BASIN.

Drainage Area above Gauging Station = 2.36 Sq. Miles.

	Rainfall.						Mean Discharge.				
(1)	(2)	(3)		(4)	(5)		(6)	(7)	(8)	(9)	(10)
Date, 1911.	Quantity, in inches.	Duration of storm.		Maximum rate, in inches.	Period of maximum rate.		Cubic feet per second.	Period, in hours.	Cubic feet per second per square mile.	Inches on water-shed.	Ratio of run-off to rainfall. Percentage.
		Hrs.	Min.		Hrs.	Min.					
April 25th.	3.12	3	15	2.70	..	..	18.75	12	7.91	0.15	5.0
May 9th...	2.08	1	55	2.06	..	..	23.03	15	9.76	0.23	11.0
May 11th..	2.97	3	15	2.45	1	00	35.40	24	15.00	0.56	19.0
May 13th..	1.50	2	35	0.61	2	10	70.80	21	30.00	1.12	72.0
May 25th..	2.73	2	4	4.04	1	00	59.20	24	25.10	0.93	34.0
May 30th..	1.25	4	30	2.85	0	38	11.77	48	4.99	0.37	29.7
July 6th...	1.45	0	50	2.60	0	32	20.82	48	8.82	0.66	45.2
July 8th...	0.98	1	10	1.61	0	32	17.76	48	7.52	0.56	57.0
Aug. 22d..	2.13	4	20	15.53	0	8	18.77	72	7.95	0.89	41.7
July 21st..	0.77	..	..	....	..	..	0.50	5	0.24	0.0039	0.21
July 23d...	0.82	1	15	1.56	0	25	2.67	12	0.88	0.016	2.00

While there are several small brooks running directly into the reservoir, most of the area is tributary to the main stream for quite a distance above high water. A short distance above this point a river stage register was established, the area tributary from above being 2.36 sq. miles, or about 78% of the total land area. There is an automatic rain gauge on the shore of the lake, about $\frac{3}{4}$ mile east of the register station. A gauging section was established near the register, and a sufficient number of gaugings were made to fix the discharge curve and rating table. In Table 40 are shown the record and distribution for some of the most severe storms which have occurred since the station was installed. Attention is called particularly to the

progressive increase in the ratio of run-off to rainfall, Column 10. Near the end of the dry season there had been only 3.34 in. of rain since February 15th, and 2.38 in. of this had been in a sudden downpour. The ground, therefore, was parched and dry, and the vegetation eager for moisture. On April 25th, 3.12 in. of rain fell in 3 hours 15 min., but only 5% appeared as run-off in the stream. As the rains became more frequent and the depleted ground storage became filled, a progressively greater quantity appeared in the stream flow until about July. A ratio of about 45% had been reached then, and the percentage of run-off afterward oscillated above and below this mark, depending on the rainfall and the quantity of water stored in the ground. The ratio of 72% appearing from the storm of May 13th is due to the heavy rainfalls just previous, which had not fully been cared for either by storage or surface flow. The data concerning the storms of July 21st and 23d are also of interest as an indication of the almost complete loss of a considerable quantity of water to stream flow and water use. This condition is due to the excessively dry period of June and July, when the rainfall was considerably below normal. It is indirect proof of the large water requirements of vegetation, and may be considered as a total loss to the lake. A study of the ground-water conditions showed that little, if any, of this rain appeared as run-off, although the run-off probably benefited indirectly by a lessened demand on the ground reservoir for vegetation.

The storm of May 25th was rather notable for the quantity of rain falling in a short time, and the conditions of run-off are those which ordinarily obtain from storms of this character. It occurred at a time when the ground reservoir was considerably filled and the demands of vegetation would not be out of the ordinary. Certain facts concerning this storm are shown by Fig. 41. The total rainfall was 2.73 in., of which a little more than 2 in. fell in 1 hour. The rain began at 12.12 P. M., and 0.29 in. fell in 26 min., no run-off being appreciable on the river gauge. The rain again began at 2.54 P. M., and at 4.32 P. M. 2.44 in. had fallen. At 3 P. M. the river gauge registered a flow of 1.3 cu. ft. per sec., and at this time the discharge began to increase. The maximum flow came at 4.15 P. M., the gauge showing a rise of 5.31 ft. and a discharge at the rate of 380 cu. ft. per sec. The stream flow did not return to its original discharge until 3 P. M. of May 30th, a period of 5 days. From 4.30 P. M. of the 25th to 3 P. M. of May 30th,

0.18 in. of rain fell between 2 A. M. and 10 P. M. of the 26th. As the 0.15 in. of rain that fell on May 25th, prior to the principal downpour, did not affect the stream flow, it is probable that the 0.18 in.

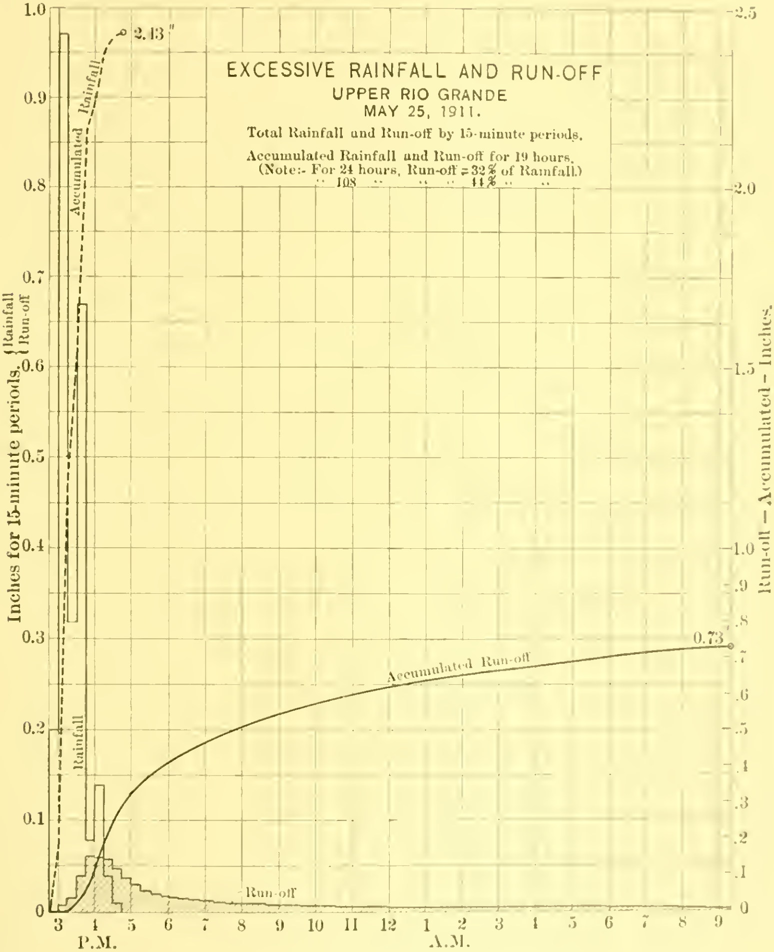


FIG. 41.

falling 9½ hours later also had so little effect as to be negligible. Assuming this condition, the rainfall between 2.54 P. M. and 4.30 P. M., May 25th, caused a run-off, which, beginning at 3 P. M., May 25th,

reached a maximum at 4.15 P. M. of the same day, and its effect was noted in the stream flow up to 3 A. M. of May 30th. That is, 2.43 in. of rain falling in 1 hour 38 min., at an average rate of 1.76 in. per hour, caused a run-off of 1.07 in. in 108 hours, at an average rate of about 0.01 in. per hour.

The ratio of discharge to rainfall was.... $\frac{1.07}{2.43} = 44$ per cent.

The ratio of time, run-off to rainfall was $\frac{1.08}{1.63} = 66.4$ " "

The flow past the gauging station showed the following quantities passing:

12% of the total rainfall and	30% of the total run-off in	2	hours.
18% " " " " "	40% " " " " "	3	"
20% " " " " "	45% " " " " "	4	"
27% " " " " "	60% " " " " "	10	"
44% " " " " "	100% " " " " "	108	"

The storm of May 25th was only an ordinary tropical rainfall covering a comparatively small area, and of common occurrence during the rainy season. It has been given in detail only because its progress was carefully watched, and is an indication of average conditions. The rise of more than 5 ft. in 1¼ hours, with a discharge increased nearly 300 times, is similar to conditions frequently encountered on watersheds of limited extent.

TABLE 41.—AREAS OF WATER-SHEDS.

DRAINAGE NORTH OF CONTINENTAL DIVIDE.		DRAINAGE SOUTH OF CONTINENTAL DIVIDE.	
Water-shed.	Square miles.	Water-shed.	Square miles.
Upper Chagres	212	Pedro Miguel	10.9
Pequini	189	Cardenas	10.6
Chilibre	54	Caimeto	18.6
Gatuncillo	38	Cocoli	12.4
La Puente	22	L. Rio Grande	10.4
Frijoles	7		
Frijolito	10.2		
Frijoles Grande	2.75		

The relation between the rainy- and dry-season discharges of streams is particularly interesting in its bearing on water supply, and the use to which it can be put in regard to stream flow and reservoir storage. Figs. 42, 43, and 44 show these conditions graphically, and

little description seems necessary. It appears that the extreme low flows last for comparatively short periods, and while in many cases the discharge of the stream through the dry months is very well sustained from ground storage, in other cases rivers draining considerable areas almost cease to flow. The areas of the several drainage basins are given in Table 41.

It is worthy of notice that the streams south of the continental divide and flowing into the Pacific Ocean generally have a much less

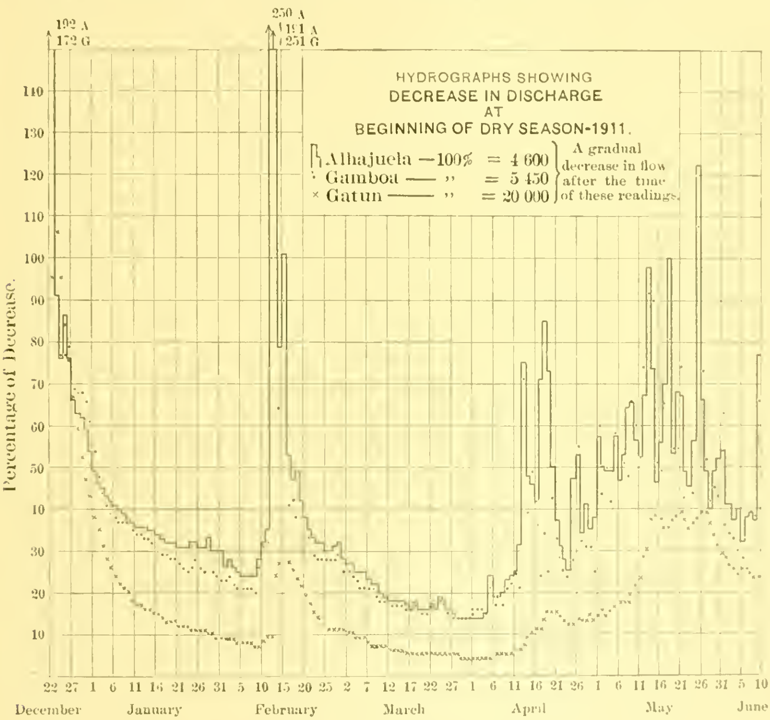
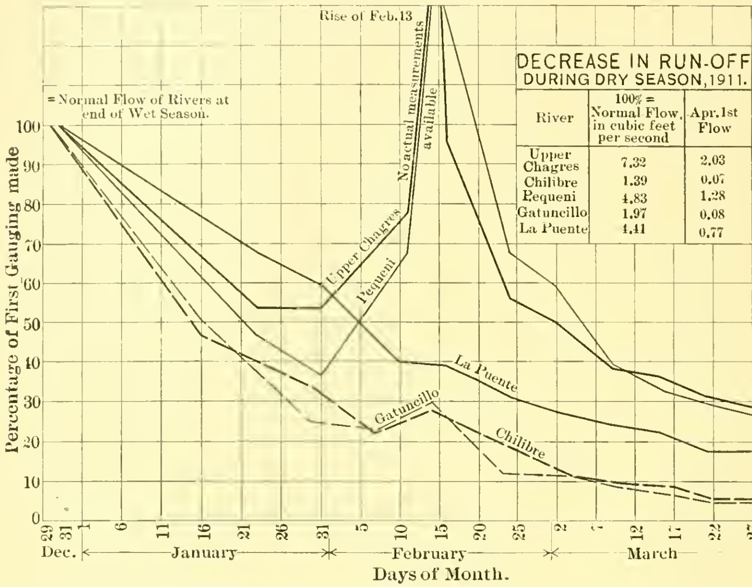


Fig. 42.

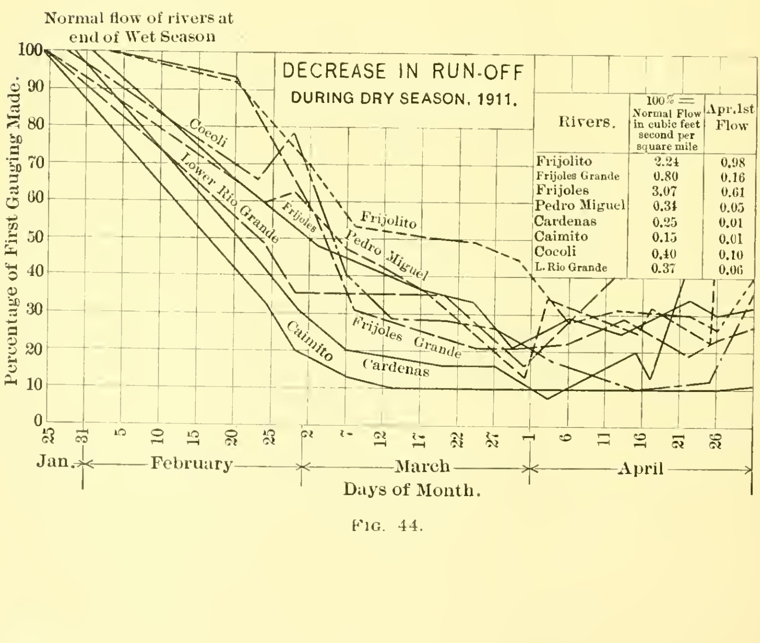
dry-season discharge than those on the northerly side. As mentioned in the discussion on rainfall, this condition is due to the prevailing winds and to the fact that the northerly side is the windward side of the cordillera.

The relations of rainfall and run-off in the basin seem to be best presented in diagrammatic form. The run-off, as measured and estimated at the several principal stations, has been given in Tables 19



River	100% = Normal Flow, in cubic feet per second	Apr. 1st Flow
Upper Chagres	7.32	2.03
Chilibre	1.39	0.07
Pequeni	4.83	1.28
Gatuncillo	1.97	0.08
La Puente	1.41	0.77

FIG. 43.



Rivers.	100% = Normal Flow in cubic feet second per square mile	Apr. 1st Flow
Frijolito	2.24	0.98
Frijoles Grande	0.80	0.16
Frijoles	3.07	0.61
Pedro Miguel	0.34	0.05
Cardenas	0.25	0.01
Caimito	0.15	0.01
Cocoli	0.40	0.10
L. Rio Grande	0.37	0.06

FIG. 44.

to 32, inclusive, and the rainfall or record in Tables 7 to 11. Figs. 45 to 50, inclusive, show the run-off in the Chagres Basin for the average year and for other critical periods at Alhajuela and Gatun. Before studying the river relations, it was necessary to have the quantity of rainfall on the area considered. An estimate of this, based on the actual records from the stations, weighted for their relative influence, have been prepared. The values are given in Tables 44 and 45.

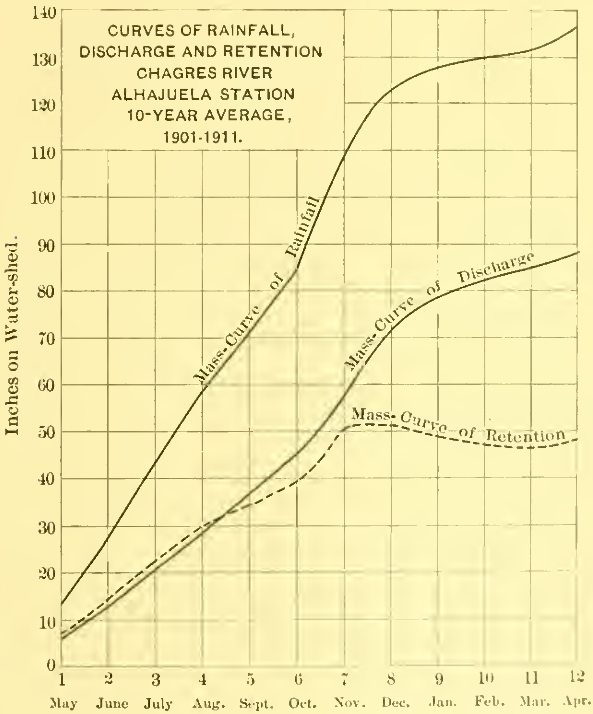


FIG. 45.

In many cases the rainfall records were lacking, and these were estimated from present annual ratios between the nearest station with a long-time record and the station where a value was required.

Attention is called to the fact that, although the run-off is a function of the rainfall, and would in time absolutely cease if the latter should stop, the quantity of the former does not vary directly with the quantity of the latter. During comparatively short periods of

little or no rainfall the run-off is many times in excess of the precipitation, while for longer periods the run-off decreases with the rainfall. This decrease, however, is not directly proportional, but is in some more complex ratio, depending on the character of the water-shed and

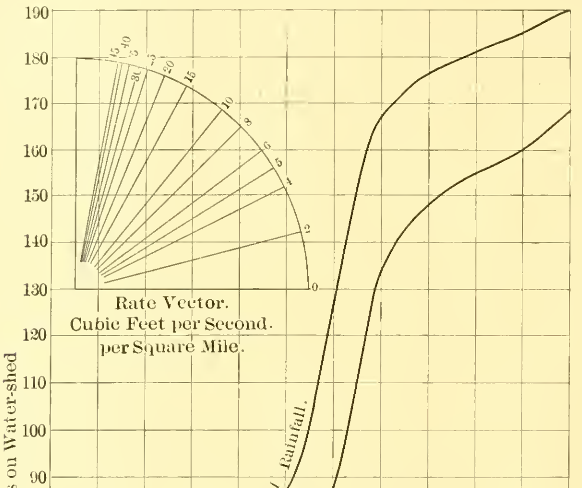


FIG. 46.

its vegetation. For example, a rainfall of 25% less than normal will produce a certain run-off during the year, under certain conditions, but a rainfall of 50% of the normal for this period would probably yield much less than 50% of the normal run-off. For this reason, it

PLATE V.
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JANUARY, 1913.
SAVILLE ON
HYDROLOGY OF THE
PANAMA CANAL.

is very unsafe to base calculations of yield on short-time observations, and apply the relation thus established to minimum conditions of rainfall existing at some former period. For the purpose of illustrating these conditions, Tables 42 and 43 and Fig. 50 are given for the whole area of the basin above Gatun (1 320 sq. miles), and for the area above Alhajuela (427 sq. miles).

The year of lowest rainfall in this decade is 1902-03, with 93.38 in. The river discharge for this year was 48.87, or 52.4 per cent.

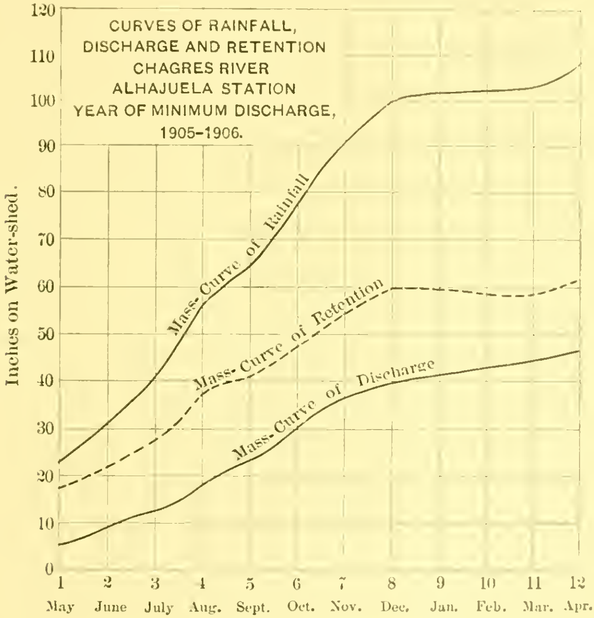


FIG. 47.

April, 1905, was quite dry, and the heavy rainfall of 20.22 in. in May produced only a moderate run-off, although it increased considerably the total rainfall for the year.

A comparison of the retention curves and discharge tables given in this paper indicates in some degree the quantity of water required by vegetation. In the year of plenteous rainfall, besides actual precipitation, the air contains more water vapor, which, being absorbed by the leaves and grasses, gives them a supply of moisture in lieu of actual rainfall. In the years of less rainfall these demands must be met from

ground storage, and thus a large quantity of actual precipitation must be taken, to the consequent detriment of stream flow.

In the year of heavy rainfall the subterranean reservoir is kept full, and rainfall reaching the surface of the ground, having no opportunity to delay, is delivered quickly to the stream with minimum loss.

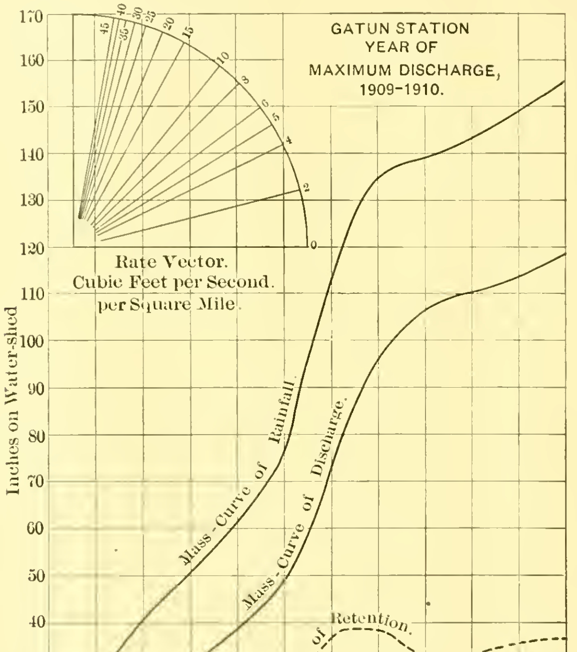
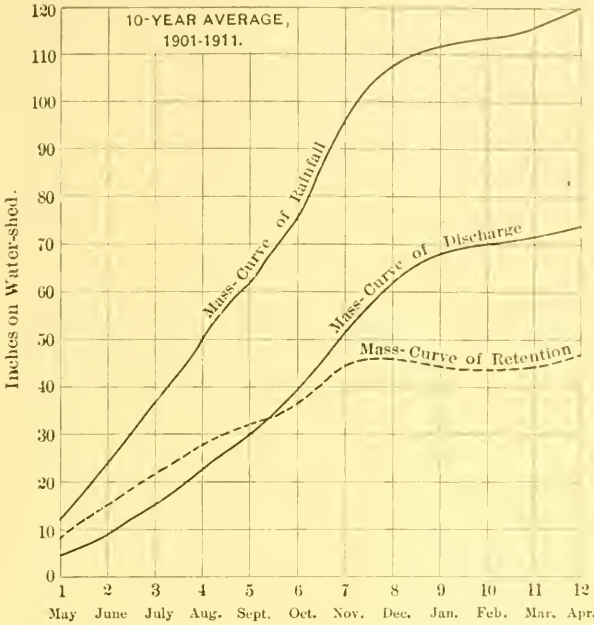


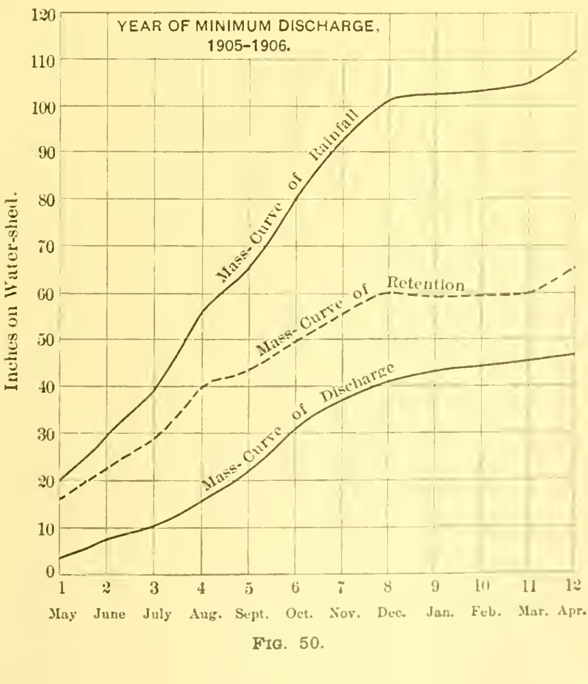
FIG. 48.

What proportion of the stream flow is due to sub-surface flow is a complex question, and each water-shed has a coefficient of its own that must be applied to any general statement. The quantity depends on the character of the soil, the underlying strata, the topography, and the surface cover of vegetation. That considerable quantities of



10-YEAR AVERAGE,
1901-1911.
Inches on Water-shed.
Mass-Curve of Rainfall
Mass-Curve of Discharge
Mass-Curve of Retention
0
10
20
30
40
50
60
70
80
90
100
110
120
1 May
2 June
3 July
4 Aug.
5 Sept.
6 Oct.
7 Nov.
8 Dec.
9 Jan.
10 Feb.
11 Mar.
12 Apr.

FIG. 49.



YEAR OF MINIMUM DISCHARGE,
1905–1906.
Inches on Water-shed.
Mass-Curve of Rainfall
Mass-Curve of Retention
Mass-Curve of Discharge
0
10
20
30
40
50
60
70
80
90
100
110
120
1 May
2 June
3 July
4 Aug.
5 Sept.
6 Oct.
7 Nov.
8 Dec.
9 Jan.
10 Feb.
11 Mar.
12 Apr.

FIG. 50.

TABLE 42.—MONTHLY RAINFALL, RUN-OFF, AND PERCENTAGE.
Chagres River Basin above Alhajuela = 427 Sq. Miles.

Month.		10-YEAR AVERAGE. 1901-11.			YEAR OF MAXIMUM DISCHARGE, 1909-10.			YEAR OF MINIMUM DISCHARGE, 1905-06.		
		Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.
May	Rainy Season.	13.08	6.01	46.0	12.00	6.54	54.5	25.04	5.21	22.6
June		13.94	6.49	46.6	18.72	12.73	68.0	8.36	4.16	49.8
July		16.03	7.76	51.6	19.98	8.91	44.6	9.08	3.37	37.2
Aug		15.36	7.95	51.8	10.99	11.04	100.2	15.78	5.96	37.8
Sept		12.56	8.08	64.3	10.74	9.31	86.7	6.00	4.50	56.3
Oct.		13.86	8.58	61.7	14.01	9.89	70.6	13.64	7.25	53.2
Nov.		23.33	12.49	53.6	40.62	29.55	72.8	13.00	5.92	45.5
Dec.		14.27	13.91	96.9	40.54	46.71	105.1	8.56	3.41	39.8
Jan.	Dry Season.	5.18	7.07	136.5	8.52	13.64	160.1	1.55	1.85	133.4
Feb		1.85	3.81	205.9	4.24	7.00	165.2	0.85	1.50	176.5
Mar		1.68	2.30	137.0	4.53	4.39	96.9	0.60	1.18	196.9
Apr		5.22	3.34	64.0	5.06	8.21	162.2	5.10	1.86	36.5
Total		136.36	87.79	64.4	189.95	167.92	88.4	107.56	46.17	42.89
8 wet months		122.43	71.27	58.2	167.60	134.68	80.4	99.46	39.78	40.00
4 dry months		13.93	16.52	118.6	22.35	33.24	148.6	8.10	6.39	78.90

TABLE 43.—MONTHLY RAINFALL, RUN-OFF, AND PERCENTAGE.
Chagres River Basin above Gatun = 1 320 Sq. Miles.

Month.		10-YEAR AVERAGE, 1901-11.			YEAR OF MAXIMUM DISCHARGE, 1909-10.			YEAR OF MINIMUM DISCHARGE, 1905-06.		
		Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.	Rainfall, in inches.	Run-off, in inches.	Percentage.
May	Rainy Season.	12.49	4.02	33.3	12.44	3.68	29.6	20.22	3.99	19.7
June		11.82	5.23	44.2	15.24	8.35	54.8	9.60	3.63	37.8
July		12.81	6.30	49.2	13.44	8.69	64.7	9.25	2.70	29.2
Aug		13.51	7.15	52.9	9.81	9.59	97.8	16.96	5.49	32.4
Sept		11.74	7.63	64.9	10.66	8.38	78.6	8.90	5.41	60.8
Oct.		13.69	8.94	65.3	14.96	11.00	73.5	15.64	9.82	62.8
Nov		20.33	12.50	61.5	35.01	24.03	69.8	12.33	6.47	52.5
Dec.		11.18	10.40	93.0	23.08	22.54	97.7	8.75	4.04	46.2
Jan.	Dry Season.	4.14	5.54	133.8	4.35	10.26	235.9	1.25	1.89	151.2
Feb.		1.69	2.26	133.8	4.64	4.01	86.4	1.06	1.10	103.8
Mar		1.96	1.46	74.5	5.61	2.76	49.2	1.15	0.87	75.7
Apr		4.70	2.13	45.3	5.96	5.14	86.2	6.83	1.37	20.1
Total		120.06	73.56	61.2	155.20	118.48	76.3	111.94	46.78	39.2
8 wet months		107.57	62.17	57.9	131.64	96.31	71.3	101.65	41.55	40.9
4 dry months		12.49	11.39	91.1	20.56	22.17	110.2	10.29	5.23	50.8

TABLE 44.—Average Estimated Monthly Rainfall, in Inches, on the Chagres River Basin.

Area of Chagres Basin = 1 320 Sq. Miles.

The yearly totals are for the run-off year beginning May 1st.

Month.	1901–1902.	1902–1903.	1903–1904.	1904–1905.	1905–1906.	1906–1907.	1907–1908.	1908–1909.	1909–1910.	1910–1911.	Monthly average.
May	10.21	11.93	12.55	10.09	22.27	10.64	7.59	19.03	11.99	11.51	12.78
June	11.06	7.89	10.72	15.16	8.90	12.37	14.90	12.24	15.12	13.03	12.14
July	10.72	14.79	16.56	12.76	9.14	16.90	12.45	9.50	14.49	18.21	13.58
August	15.50	8.88	14.31	9.73	17.84	18.61	14.72	16.88	9.54	11.06	14.01
September	13.91	11.75	11.30	14.03	8.88	12.97	11.17	10.31	10.76	16.11	12.12
October	16.19	14.37	12.39	8.72	16.10	12.81	17.61	12.01	15.09	14.37	13.97
November	29.20	16.09	23.24	20.89	12.81	22.82	11.85	18.80	35.74	20.69	21.21
December	6.29	6.66	17.27	5.82	9.00	15.35	4.78	8.96	26.20	19.44	11.98
January	12.90	1.07	5.81	6.49	1.32	1.92	2.23	8.01	5.14	0.50	4.54
February	0.58	0.27	1.01	1.07	1.06	1.40	0.78	4.59	4.94	2.70	1.81
March	1.91	0.79	1.66	0.67	1.21	1.96	1.86	2.45	5.97	1.31	1.98
April	4.74	1.46	11.09	1.90	6.39	0.90	3.05	8.44	6.16	4.77	4.89
Year	133.21	95.95	137.91	107.33	115.22	128.65	102.99	131.22	161.14	136.73	125.01

The average monthly rainfall over the Chagres River Basin was computed from the following records: The records from each section are given weights corresponding to the area of the section.

Section.	Rainfall Records.	Weight.
No. 1	Porto Bello + Alhajuela ÷ 2	427
No 2	Alhajuela + Gamboa ÷ 2	132
No. 3	Culebra + Gamboa ÷ 2	32
No. 4	Gamboa + Bohio ÷ 2	188
No. 5	Trinidad	314
No. 6	Monte Lirio	127
No. 7	Bohio + Gatun ÷ 2	100
Total		1 320

TABLE 45.—Average Monthly Rainfall, in Inches, on the Chagres River Basin above Alhajuela.

Area = 427 Sq. Miles.

Month.	(1901)	(1902)	(1903)	(1904)	(1905)	(1906)	(1907)	(1908)	(1909)	(1910)	Monthly average.
May	11.71	12.42	12.90	10.57	23.04	10.98	6.38	20.38	12.00	10.40	13.08
June	12.74	8.80	11.93	19.00	8.36	14.28	17.14	11.02	18.72	14.44	13.91
July	10.01	16.65	20.05	15.92	9.08	21.32	17.54	9.83	19.98	19.90	16.03
August	16.57	8.25	16.56	10.52	15.78	21.22	16.86	16.81	10.99	17.07	15.36
September	15.20	11.81	11.25	15.76	8.00	12.94	13.09	11.02	10.74	15.80	12.56
October	18.86	14.32	13.88	9.18	13.64	12.26	18.86	11.18	14.01	12.46	13.86
November	30.72	16.76	25.41	23.22	13.00	25.94	11.54	26.14	40.62	18.98	23.33
December	5.74	6.59	19.96	5.83	8.56	14.91	6.46	14.87	10.54	19.19	14.27
	(1902)	(1903)	(1904)	(1905)	(1906)	(1907)	(1908)	(1909)	(1910)	(1911)	
January	12.36	0.71	5.78	6.21	1.55	1.55	2.85	11.81	8.52	0.49	5.18
February	0.54	0.30	0.40	0.74	0.85	0.86	0.75	5.26	4.24	4.60	1.85
March	1.62	0.78	1.30	1.18	0.60	1.41	2.28	1.92	1.53	1.20	1.68
April	5.55	1.24	10.47	2.12	5.10	0.76	3.08	13.05	5.06	5.74	5.22
Year	141.65	98.63	149.89	120.25	107.56	142.46	116.83	156.29	189.95	140.27	135.36

The monthly values are equal to the mean of the rainfall records at Alhajuela and Porto Bello.

The annual totals are for the run-off year beginning May 1st.

TABLE 46.—(*Continued.*)

April 6..	0.22	+ 4.01	5.65	4.23	1.42	1.64	0.170	0.52	25.1	3.9	
April 12..	0.70	+ 2.77	5.84	3.47	2.37	3.07	0.224	2.38	40.6	12.0	
April 18..	0.39	+ 4.10	5.83	4.49	1.34	1.73	0.126	0.05	23.0	6.7	$\frac{0.958}{6.76}$ = 14.2%
April 24..	0.89	+ 4.20	5.62	5.09	0.53	1.42	0.103	0.31	9.4	15.8	
April 30..	2.40	+ 0.65	5.93	3.05	2.88	5.28	0.385	3.50	48.5	40.5	
May 6..	0.50	+ 2.50	5.83	3.00	2.83	3.33	0.243	2.24	48.5	8.6	
May 12..	7.04	− 13.29	5.61	− 6.25	11.86	18.90	1.377	5.23	211.2	125.5	
May 18..	13.32	− 11.60	5.32	1.72	3.60	16.92	1.233	2.25	67.7	250.3	$\frac{6.086}{16.65}$ = 36.6%
May 24..	6.06	− 5.01	5.25	1.05	4.20	10.26	0.747	2.75	80.0	115.4	
May 30..	11.11	− 27.50	6.62	− 16.39	23.01	34.12	2.486	4.18	347.4	167.8	
June 5..	3.71	− 1.91	5.55	5.62	0.93	4.64	0.338	0.53	14.2	56.6	
June 11..	0.93	+ 2.00	5.40	2.93	2.47	3.40	0.248	1.04	45.7	17.2	
June 17..	0.54	+ 2.15	4.66	2.69	1.97	2.51	0.183	0.54	42.3	11.6	$\frac{1.212}{4.27}$ = 28.4%
June 23..	0.15	+ 3.11	5.09	3.26	1.83	1.98	0.144	0.54	36.0	2.9	
June 29..	1.67	+ 0.83	4.94	2.50	2.44	4.11	0.300	1.62	49.4	33.8	
July 5..	1.48	+ 1.27	4.99	2.75	2.24	3.72	0.271	1.26	44.9	29.7	
July 11..	10.84	− 10.51	6.19	0.33	5.86	16.70	1.217	2.49	94.7	175.2	
July 17..	1.37	+ 2.71	6.12	1.09	2.03	3.40	0.248	0.19	33.2	22.4	$\frac{2.205}{6.67}$ = 33.1%
July 23..	0.50	+ 2.60	5.48	3.10	2.38	2.88	0.210	2.06	43.4	9.1	
July 29..	0.74	+ 1.83	5.38	2.55	2.83	3.57	0.260	0.67	52.6	13.4	
Aug. 4..	5.41	− 6.81	5.90	− 1.40	7.30	12.71	0.927	2.66	123.8	91.7	
Aug. 10..	2.26	+ 1.44	6.44	3.70	2.74	5.00	0.365	1.44	42.5	35.1	
Aug. 16..	1.84	+ 2.22	5.46	4.06	1.40	3.24	0.236	0.99	25.6	33.7	$\frac{2.273}{8.16}$ = 27.9%
Aug. 22..	3.19	− 0.94	5.20	2.25	2.95	6.14	0.448	2.27	56.7	61.3	
Aug. 28..	2.61	+ 1.38	5.45	3.99	1.46	4.07	0.297	0.80	26.8	47.9	
Sept. 3..	2.89	+ 0.39	5.19	3.28	1.91	4.80	0.350	1.70	36.8	55.6	
Sept. 9..	6.79	− 4.16	5.56	2.63	2.93	9.72	0.709	1.69	52.7	122.1	

* Constant (0.0729): - c. f. s. to inches on water-shed (3.06 sq. mi.) for 6-day period.
† The mean discharge in inches for the monthly period divided by the total rainfall in inches.

TABLE 46.—Rio Grande Reservoir Data.
Elevation of Spillway = 235.2.

Supply:
Main stream...... }
Brooks............ } = { Consumption (V. meter).
Ground-water..... } { Evaporation (4-ft. pan).
Flood storage..... } { Waste (weir discharge).
{ Seepage.

Water-shed:	Sq. Mi.
Total area........................	3.15
Water area........................	0.09
Land above gauge station.....	2.36
Land below gauge station......	0.70

Period ending.	Supply.		Discharge.			Run-off Surface + Ground Flow:			Percentages.		
	Surface flow, in cubic feet per second.	Storage, in cubic feet per second. + Decr. − Incr.	Consumption, Evaporation + Weir, in cubic feet per second.	Surface + storage, in cubic feet per second. Columns (2) + (3).	Ground-water, in cubic feet per second. Columns (4) − (5).	in cubic feet per second. Columns (2) + (6).	in inches. Column 7 × 0.0729.*	Rainfall, in inches.	Ground-water to total discharge. Columns (6) − (4).	Surface supply to total discharge. Columns (2) − (4).	Run-off to rainfall. Columns (8) − (9).†
Mean Discharge.											
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Jan. 30..	2.65	+ 3.66	6.58	6.31	0.27	2.92	0.213	0.00	4.1	40.3	
Feb. 5..	1.93	+ 3.49	6.30	5.42	0.88	2.81	0.205	0.03	14.0	30.6	0.945/0.88 = 107.4%
Feb. 11..	1.69	+ 3.43	5.85	5.12	0.73	2.42	0.176	0.39	12.5	28.9	
Feb. 17..	1.62	+ 3.16	5.77	4.78	0.99	2.61	0.190	0.46	17.2	28.1	
Feb. 23..	1.24	+ 3.48	5.75	4.72	1.03	2.27	0.161	0.00	17.9	21.6	
Mar. 1..	0.93	+ 3.56	5.38	4.49	0.89	1.82	0.133	0.03	16.5	17.3	
Mar. 7..	0.91	+ 4.08	5.53	4.99	0.54	1.45	0.106	0.00	9.8	16.5	
Mar. 13..	0.91	+ 4.15	6.06	5.06	1.00	1.91	0.139	0.00	16.5	15.0	0.650/0.09 = 723%
Mar. 19..	0.89	+ 4.41	5.77	5.30	0.47	1.36	0.099	0.00	8.1	15.4	
Mar. 25..	0.61	+ 4.46	5.70	5.07	0.63	1.24	0.090	0.06	11.1	10.7	
Mar. 31..	0.32	+ 4.50	5.64	4.82	0.82	1.14	0.083	0.00	14.5	5.7	

water are received in this way is an undisputed fact. Table 46 shows some conditions that have obtained on the Rio Grande water-shed. No attempt is made to discuss the subject in this paper, the intention being to make further study before presenting conclusions. The data thereon, however, are applicable to some extent to conditions of the proposed Gatun Lake, and indicate a considerable increase in supply other than that due to surface conditions. Figs. 45 to 50, inclusive, present graphically the conditions of rainfall, run-off, and retention which obtain in the basin above Alhajuela and over the entire drainage area above Gatun. The large quantity unaccounted for in minimum years, and the relatively small quantities "lost" in years of maximum rainfall, are, of course, what would be expected.

Hydrography of Gatun Lake and the Panama Canal.

Gatun Lake, which will be formed by the big earth dam across the Chagres River at Gatun, is one of the largest artificial bodies of water in the world, if not the largest. There is no similarly made body of its size in the Western Hemisphere, the nearest approach being the proposed Bear Lake Reservoir, in Utah, with a surface of 108 sq. miles, and it is probable that Egypt alone in the Eastern Hemisphere has a lake comparable either in area or capacity.

TABLE 47.—Areas and Volumes of Gatun Lake.

Contour. (Feet above mean sea level).	Area.		Volume.		
	Millions of square feet.	Square miles.	Millions of cubic feet.	Millions of gallons.	Acre-feet.
0	12	0.44	......	......	......
10	385	13.81	1 982	14 825	45 499
20	813	29.17	7 972	59 637	183 031
30	1 426	51.14	19 169	143 385	440 062
40	1 971	70.69	36 152	270 419	829 942
50	2 529	90.71	58 651	438 707	1 346 434
60	3 099	111.17	86 791	649 198	1 992 450
70	3 670	131.65	120 638	902 375	2 769 476
80	4 274	153.32	160 361	1 199 507	3 681 401
85	4 555	163.38	183 136	1 369 859	4 204 228
90	4 835	173.44	205 911	1 540 211	4 727 056

At Elevation + 87 Gatun Lake has an area of 167.4 sq. miles, of which about 90 sq. miles are within the Canal Zone and the remainder in Panamanian territory. The shore line, not including islands, is about 1 010 miles in length, of which about two-thirds are without the Zone.

Table 48* is given for purposes of comparison. The reservoirs listed therein are either built or projected.

TABLE 48.

Name.	Capacity, in thousands of millions of cubic feet.	Name.	Capacity, in thousands of millions of cubic feet.
GATUN LAKE	205.91	New Croton Dam, N. Y.	7.85
Cataracte de Assouan, Egypt	130.65	Buena Vista Lake, Cal.	7.41
Au Sud de Philos, Egypt	126.40	Penai, India	6.90
Gebel Silsileh, Egypt	123.60	Hemet Valley, Cal.	6.00
Cataracte de Assouan, Egypt	94.40	Lake McMillan, N. Mex.	6.00
Kalabehele	90.40	Bhatgus, India	5.48
Gebel Silsileh, Egypt	84.50	Paragon, India	5.29
Swan Valley Lake, Idaho	65.30	Laramie River Dam, Wyo.	5.24
Ouady Rayan, Egypt	35.31	Lake Fife, India	4.91
Tonto Basin, Ariz.	32.00	Indian River, N. Y.	4.35
Belle Fourche, S. D.	9.35	Jarpen, India	2.69
Wachusett Dam, Mass.	8.40		

Total area, Gatun Lake Drainage Basin.....	1 320	sq. miles.
Land " (Lake surface El. 87 M. S. L.)..	1 152.6	" "
Water " " " " 87 " " " ..	167.4	" "
Ratio of water to land surface............	14.52	per cent.

The top of the upper miter-sill at the locks has an elevation of + 37.33 above mean sea level, and the bottom of Culebra Cut will be at Elevation + 40, referred to the same base. The largest vessel yet built is the *Olympic,* which, when loaded to full capacity of 60 000 tons, is said to have a draft of 37.5 ft. If the minimum lake level is assumed at Elevation + 80.33, this vessel would have about 5 ft. of water under her keel when passing over the miter-sills and more than 2 ft. when passing through Culebra Cut. The heaviest United States war vessel is the *Utah,* recently in commission, and she, when loaded, is said to have a draft of 28.5 ft.; that is, at the minimum lake level assumed above, the *Utah* could pass through Culebra Cut with at least 11 ft. of water under her keel.

A storage depth of 6.67 ft., or 80.04 in., is equivalent to 11.624 in. distributed over the land area at Elevation + 87; that is, a run-off of 11.624 in. from the total land area, would raise the surface of the lake 80.04 in. This quantity, therefore, is required to make good the loss in storage when the lake surface is drawn down to the minimum, or

* Abstracted in part from Wilson's "Manual of Irrigation Engineering," Schuyler's "Reservoirs for Irrigation, Water Power and Domestic Water Supply," Buckley's "The Irrigation Works of India," and Barrois' "*Les Irrigations en Egypte.*"

the reservoir will stand this much depletion without reaching a surface level below that taken as a minimum. Reducing to equivalents, 1 in. on the land surface equals 0.574 ft. of lake storage. It is evident that, after considering the several unavoidable and continuous losses, such as maintenance, power, seepage, etc., the remainder is the quantity available for lockage.

TABLE 49.

Condition.	Elevation.	Area, in square miles.	Storage capacity in cubic feet.
Extreme high water	+87.00	167.4	192 246 000 000
Extreme low water	+80.33	154.0	161 864 000 000
Difference	6.67	13.4	30 382 000 000

The continuous draft or loss to lake storage was placed by the original Consulting Board of Engineers as:

Power, light, etc.	275	cu. ft. per sec.
Leakage at gates	275	" " " "
Seepage and other losses	85	" " " "
	635	" " " "

and these figures have been used in the following investigation, although it has been stated that recent experiments on the Stoney gates in place have demonstrated that the leakage on that account was exceedingly small, and it is probable that the figures given above will be greatly in excess of actual conditions.

The total requirements for seepage, maintenance, leakage, etc., of 635 cu. ft. per sec. are equivalent to 3.793 cu. ft. per sec. per sq. mile of lake area, which, converted into inches, gives the results in Table 50 for the several months.

TABLE 50.

Month.	Equivalent Depth, in Inches.	
	Lake area.	Land area
28-day	3.95	0.574
29-day	4.09	0.594
30-day	4.23	0.614
31-day	4.37	0.634

The rainfall on Gatun Lake surface has been estimated from the records at the stations in this area, and gaps in the record have been filled by using the observed ratios between the given stations and those in the vicinity where the records were continuous. After completing the monthly periods, from 1901 to 1911, inclusive, for the several stations to be used, the quantities from these stations were given proper weight in making up the final table.

TABLE 51.—ESTIMATED AVERAGE MONTHLY RAINFALL, IN INCHES, GATUN LAKE SURFACE.

Lake Area at Elevation + 87 = 167 Sq. Miles.

Month.	1901.	1902.	1903.	1904.	1905.	1906.	1907.	1908.	1909.	1910.	Monthly mean.
May	10.62	11.73	12.49	10.67	22.72	11.09	8.56	18.14	11.83	12.96	13.08
June	11.27	6.33	9.88	12.63	9.18	11.77	14.30	13.15	13.51	11.83	11.38
July	12.51	11.73	14.15	11.35	9.56	14.62	9.76	9.24	11.42	18.08	12.24
August	16.57	6.78	12.82	9.61	19.24	14.55	13.36	17.85	9.35	13.16	13.33
September	15.91	11.02	12.32	13.06	9.97	13.54	10.36	9.62	11.17	15.65	12.26
October	16.22	14.56	11.44	8.49	17.78	13.50	18.04	12.64	15.52	15.29	14.35
November	31.15	12.50	23.43	19.93	13.94	20.65	12.62	16.87	34.84	22.76	20.87
December	7.16	5.78	16.15	6.08	9.33	16.00	4.20	6.08	21.12	20.18	11.21
January	19.07	1.68	6.38	7.51	1.22	2.33	2.14	6.83	3.92	0.59	5.17
February	0.63	0.28	1.59	1.39	1.39	1.80	0.97	1.15	5.44	1.86	1.95
March	2.12	0.94	2.11	0.34	1.58	2.47	1.93	3.04	7.31	1.51	2.34
April	5.22	1.98	12.01	1.56	6.99	1.01	2.59	6.31	6.79	4.57	4.91
Year	148.45	85.31	134.77	102.62	122.90	123.33	98.83	123.95	152.22	138.44	123.09

The yearly totals are for the run-off year beginning May 1st. The average monthly rainfall over Gatun Lake was computed from the following records:

Station.	Weight.
Monte Lirio	11
Trinidad	53
Bohio + Alhajuela ÷ 2	42
Bohio + Gatun ÷ 2	61

The lowest annual rainfall of record at Colon was 86.50 in., during the calendar year of 1884. The rainfall at Gamboa for the same year was 96 in.

The weights given are the numbers corresponding to the area of lake surface that appeared to be represented by this particular station. It may be noted that the average of the Bohio and Alhajuela rainfalls of record is very close to that of Gamboa, which is nearly in the center

of this district. It is realized that nothing is added to the accuracy of the records by the interpolations for supplying missing data, either in rainfall or run-off tables. On the other hand, no more reliable data are at hand, and the intention in extending these tables has been to afford a basis for comparison extending over a similar term of years in all cases. The tabulations thus made are given as the best available, in the judgment of the writer, and do not purport to be a record of conditions that have existed. From close personal observation for the past 4 years, the writer is of the opinion that the results which are based on these estimates are within reasonable limits of error.

Besides the rainfall and the unavoidable demands for water, there is a very considerable loss to storage due to evaporation. The records and a discussion of this phenomenon have been given previously. From

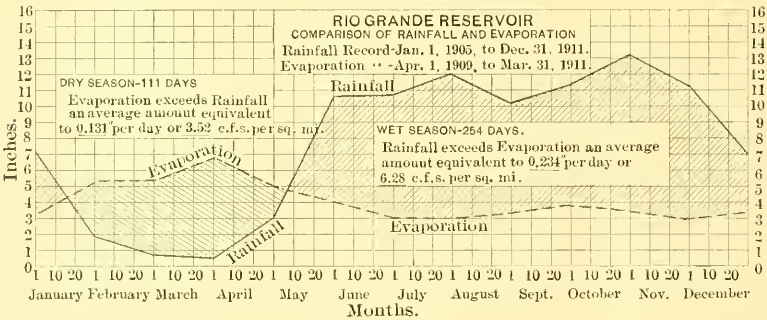


FIG. 51.

these records, the diagrams on Plate VI and Figs. 50 to 52, inclusive, have been prepared to show the relations that exist between rainfall and evaporation on water surfaces on the Isthmus, and also to show an application to the surface of Gatun Lake. In many of the estimates there has been no allowance for the gain due to rainfall on the lake surface. The evaporation figure usually has been taken as equivalent to maximum conditions, and this has been applied flat throughout the year without reference to the rainfall. Fig. 50 and Plate VI give the actual conditions which existed during the calendar year, 1910, on Rio Grande and Brazos Brook Reservoirs. Fig. 51 shows the estimated evaporation from the surface of Gatun Lake had it been in existence during 1902-03, the year of minimum rainfall at Cristobal and that of estimated minimum on Gatun Lake.

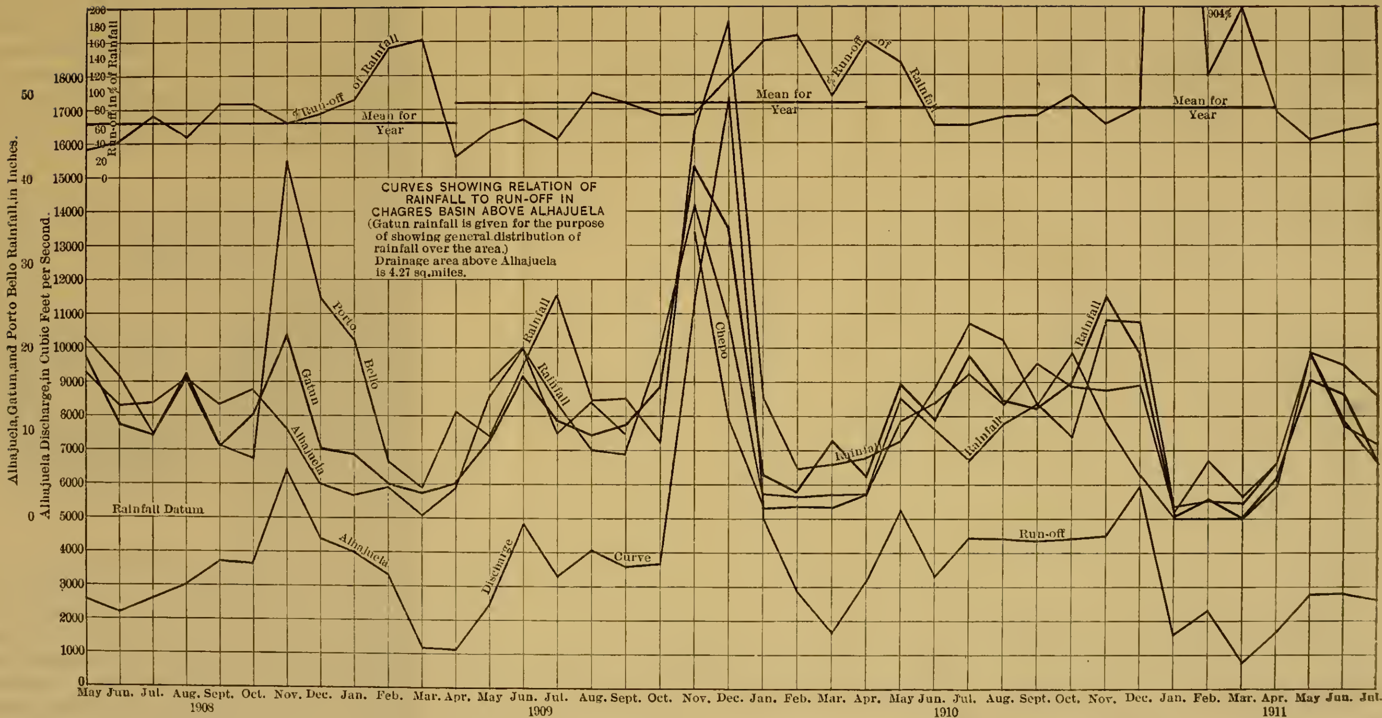

CURVES SHOWING RELATION OF RAINFALL TO RUN-OFF IN CHAGRES BASIN ABOVE ALHAJUELA
(Gatun rainfall is given for the purpose of showing general distribution of rainfall over the area.)
Drainage area above Alhajuela is 4.27 sq.miles.
Albajuela,Gatun,and Porto Bello Rainfall,in Inches.
Alhajuela Discharge,in Cubic Feet per Second.
Run-off in % of Rainfall
% Run-off of Rainfall
Mean for Year
Porto Bello
Gatun
Alhajuela
Chepo
Rainfall
Rainfall Datum
Alhajuela Discharge Curve
Run-off
904%
May Jun. Jul. Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May Jun. Jul. Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May Jun. Jul. Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May Jun. Jul.
1908
1909
1910
1911

Fig. 52 shows the probable average conditions which would have existed on Gatun Lake during the past 10 years, had the lake been then in existence. These diagrams, together with that on Fig. 8, appear to indicate that the period of excessive daily evaporation is extremely limited, and that the apparent considerable losses to water surface during the so-called dry months are in reality greatly modified by the rainfall, which is especially persistent on the Gatun Lake area during this period. For investigations concerning water supply and storage conditions, the so-called mass-diagram presents a very satisfactory and simple solution of many problems, and a study of the character of the various curves brings out facts which might otherwise be overlooked. Desmond

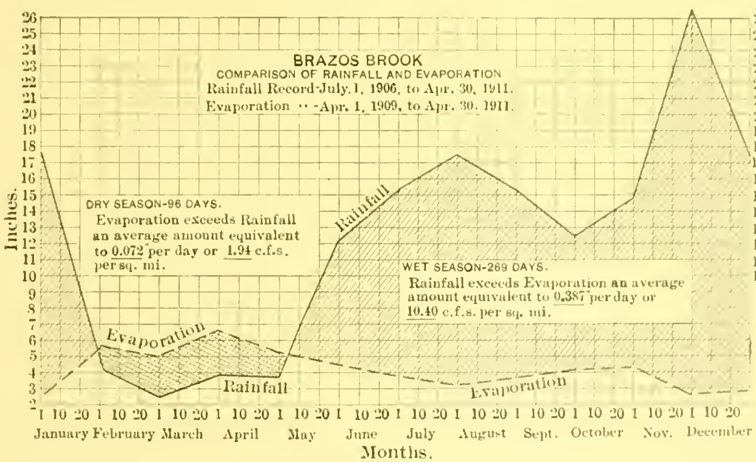


FIG. 52.

FitzGerald, Past-President, Am. Soc. C. E., was probably the first to use this method in the United States, and it appears in his report on the "Capacity of the Sudbury River and Lake Cochituate Water-Sheds in Time of Drought," 1887. Mr. FitzGerald credits the use of this diagram to Mr. W. Rippl.* John R. Freeman, M. Am. Soc. C. E., has made the most extended use of this method in working up the probable yield of the Croton water-shed.† A very clear description is given therein of the construction and properties of this curve, and Theodore Horton, M. Am. Soc. C. E.,‡ has gone into the matter in

* *Minutes of Proceedings*, Inst. C. E., Vol. 71, p. 270.

† "Report on New York's Water Supply," Freeman, 1900, pp. 226–228.

‡ *Engineering Record*, July 31st, 1897.

considerable detail. Briefly, the mass-curve is the accumulated run-off of the drainage area (in this paper reduced to flow per square mile of land area) corrected for the rainfall and evaporation on the water surface, if a pond or reservoir is included in the area, and including the known constant withdrawals. Given this mass-curve of supply and the available storage capacity of the reservoir reduced to the same unit as the mass-curve, it is readily seen that, by projecting straight lines representing uniform rates of flow, it is possible to read directly from the diagram either the maximum draft that can be counted on during various seasons, or to find what will probably be the reservoir's depletion under various drafts. To quote Mr. Freeman:

"To find from this mass-curve the periods of greatest storage depletion and the dry year cycles of longest duration, we first lay off on the diagram straight lines at slopes corresponding on the scale of the diagram to various rates of uniform draft per square mile per day, and then by straight-edge and triangle, rule parallel lines tangent to various summits, corresponding to full reservoirs, * * *. The vertical space between this straight line and the curve now represents the depletion of the reservoir under the given rate of draft; and the distance from *A* to *E* [the horizontal distance from the summit or origin of the rate line to the point where this line cuts the mass-curve.—C. M. S.] represents the length of time that the reservoir would remain drawn down."

Such a curve has been constructed for the drainage area of Gatun Lake for the period, 1901 to 1911, inclusive, and is shown by Plate VII.

In order that its construction may be clearly understood, the computations for the year 1902-03 are given in Table 52.

In Table 52: Column 2 is the run-off past Gatun (see Table 30). Column 3 is the estimated rainfall on the Lake (see Table 51). Column 4 is the mean evaporation for the past 2 years at Brazos Brook Reservoir, which, on account of its location, appeared to approximate most closely the conditions on Gatun Lake. Column 5 is the constant draft of 635 cu. ft. per sec., reduced to an equivalent in inches on the lake surface. Column 7 is the yield of the water surface of the lake in terms of the land surface; that is, Column 6 $\times \left(\frac{167.4}{1\,152.6}\right) = 14.52$ per cent.

TABLE 52.

(1) Months 1902–03.	(2) Run-off or rainfall collected, in inches.	(3) Rainfall on Gatun Lake, in inches.	(4) Evaporation on Gatun Lake, in inches.	(5) Constant Draft. Use and Loss; Power Leakage and Seepage, in inches.	(6) Algebraic sums Columns 3+4+5, in inches.	(7) Yield of water surface in terms of land surface 14.52%—Column 6, in inches.	(8) Total yield, algebraic sum of Columns 2 + 7, in inches.	(9) Accumulated yield, in inches.
Nov	9.14	12.50	2.44	4.23	5.83	0.85	9.99	132.25
Dec........	4.25	5.78	2.56	4.37	—1.15	—0.17	4.08	136.33
Jan.......	2.29	1.68	5.46	4.37	—8.15	—1.18	1.11	137.41
Feb........	1.31	0.28	4.89	3.95	—8.56	—1.24	0.07	137.51
Mar	0.90	0.94	6.51	4.37	—9.94	—1.44	—0.54	136.97
Apr	0.77	1.98	5.20	4.23	—7.45	—1.08	—0.31	136.66
May..... .	2.14	12.49	4.45	4.37	3.67	0.53	2.67	139.33
June......	3.48	9.88	3.66	4.23	1.99	0.29	3.77	143.10
July.......	5.70	14.15	3.03	4.37	6.75	0.98	6.68	149.78
Aug.......	7.75	12.82	3.47	4.37	4.98	0.72	8.47	158.25
Sept.......	7.79	12.32	3.99	4.23	4.10	0.59	8.38	166.63
Oct..	7.89	11.44	4.17	4.37	2.90	0.42	8.31	174.94

The quantity for November in Column 9 is the sum of Column 8 from May, 1901.

The year, 1902-03, was taken as an example because its dry-season discharge was one of the lowest of the decade (1908 being about the same), and it was also the year of minimum rainfall in this period.

In order to compare quickly the given supply with assumed drafts at uniform rates, the vector diagram is added, with which it is a very simple matter to obtain the required information regarding storage, depletion, and length of time the lake will be below high water.

In his discussion on the "Water Supply for the Lock Canal at Panama,"* Col. H. F. Hodges states:

"With these assumptions [previously given.—C. M. S.], and taking into consideration the saving in water which can be effected by cross-filling, the total average consumption for one passage per day, *i. e.*, for two prisms of lift each day, is 44.2 cu. ft. per sec. Neglecting any advantage to be derived from cross-filling, it is 51.9 cu. ft. per sec."

In the following computation regarding the number of lockages possible per day, as far as water supply is concerned, it is taken as 51.9 cu. ft. per sec., or about 0.04503 cu. ft. per sec. per sq. mile of land area.

For converting cubic feet per second per square mile into inches

* *Transactions*, Am. Soc. C. E., Vol. LXVII, pp. 105-106.

depth on the tributary area, the following relations are convenient, 1 cu. ft. per sec. per sq. mile being equivalent to:

Period in days.	Inches.	Logarithms.
1	0.03719	8.5704262
28	1.04132	0.0174507
29	1.07851	0.0330214
30	1.11570	0.0476642
31	1.15289	0.0618292
365.25	13.58380	1.1330164

The factors in the construction of the rate diagram are given in Table 53.

TABLE 53.

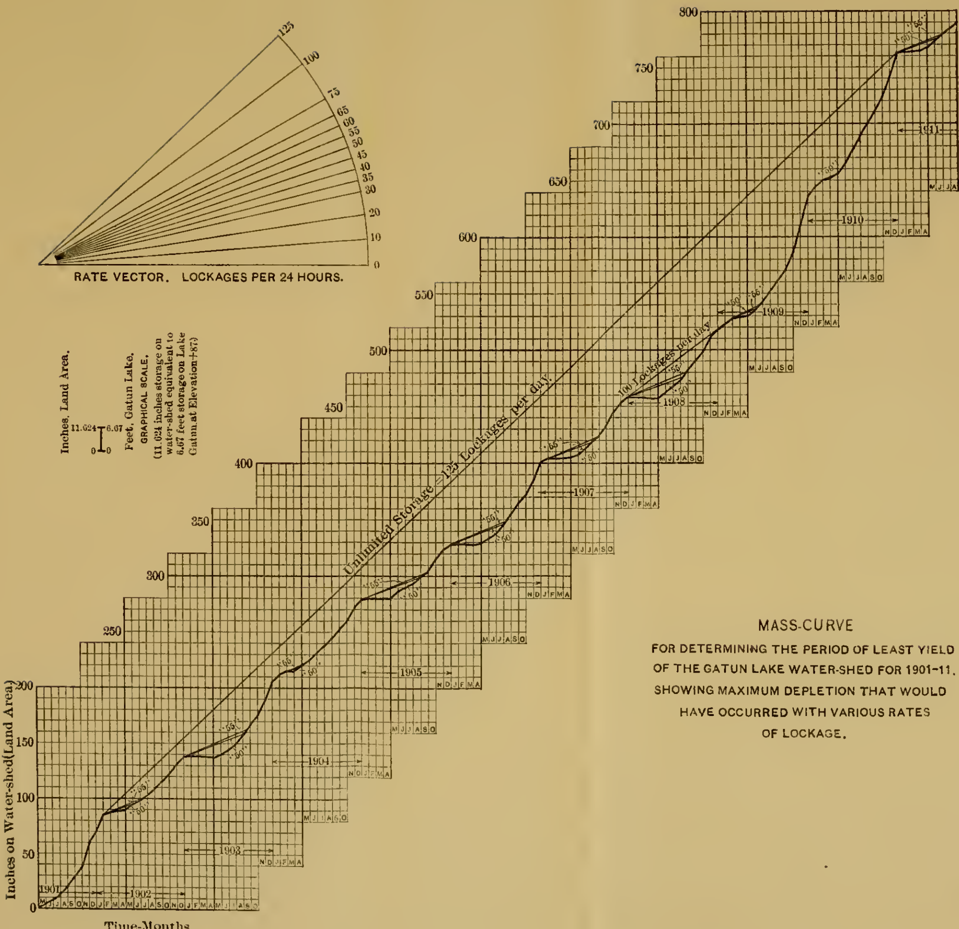
Number of lockages per day.	Cubic Feet Draft per Second.		Equivalent depth per year, in inches. Land area.
	Total.	Per square mile of land area.	
55	2854.5	2.476	33.635
50	2595.0	2.251	30.577
45	2335.5	2.026	27.519
40	2076.0	1.801	24.461
35	1816.5	1.576	21.403
10	519.0	0.4502	6.115

From inspection of Plate VII it is evident that the critical periods, as far as storage is concerned, are to be found in 1903 and 1908. In order to study these conditions better, the diagram, Fig. 55, has been prepared covering these same periods but on a larger scale.

The construction of these diagrams is such that any measurement on the vertical scale represents inches depth per square mile of land area, and any measurement on the horizontal scale represents time in months. It is also to be remembered that the available lake storage is equivalent to 11.624 in. over the land surface. Therefore, direct measurement of the number of lockages* per day that can be made is possible, considering that the lake is full at the beginning of each dry season, and is not to be drawn below Elevation 80.33. The lake will be full at the beginning of the dry season if the line representing the required number of lockages does not in any year pass above the yield summit of the next following year.

* Wherever the word "lockage" is used in this paper, it means one complete passage of the canal; that is, the expenditure of water required for passing a vessel from one ocean up one slope and down the other to the ocean on the opposite side of the Isthmus.

PLATE VII.
PAPERS, AM. SOC. C. E.
JANUARY, 1913.
SAVILLE ON
HYDROLOGY OF THE
PANAMA CANAL.



MASS-CURVE

FOR DETERMINING THE PERIOD OF LEAST YIELD OF THE GATUN LAKE WATER-SHED FOR 1901-11. SHOWING MAXIMUM DEPLETION THAT WOULD HAVE OCCURRED WITH VARIOUS RATES OF LOCKAGE.

In the case of the left-hand diagram of Fig. 58, the distance, 11.624 in., measured up from *F*, comes just above the line, *BC*, representing 55 lockages per day; and in the case of the right-hand diagram on Fig. 58, the distance up from *K* is a little above the line, *GH*, representing 55 lockages per day. Therefore, 55 lockages per day is

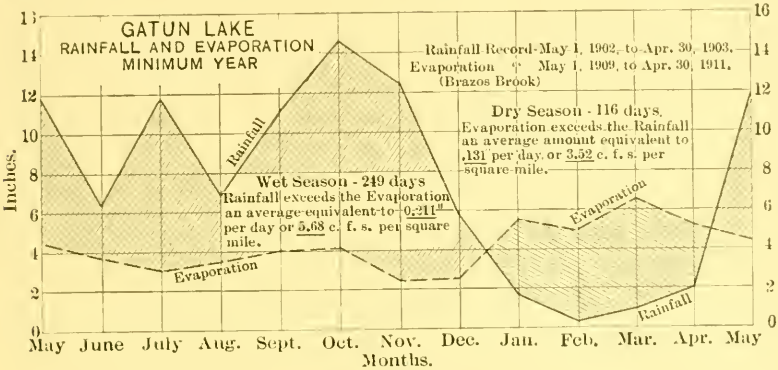


FIG. 53.

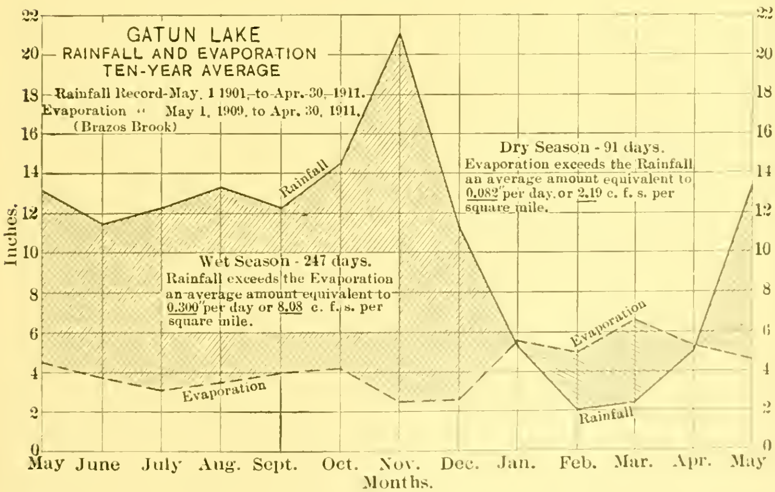


FIG. 54.

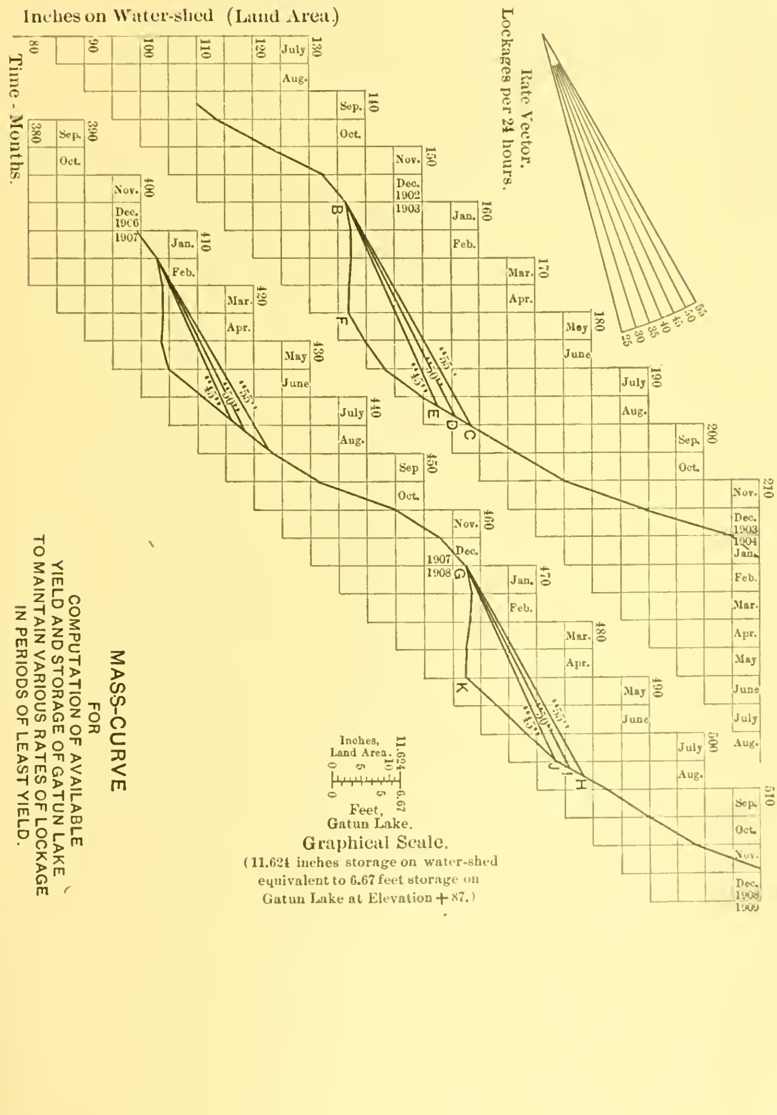
the maximum that can safely be counted on with present conditions of storage. On the horizontal scale each small subdivision represents a month. In the left-hand diagram, the line, *BC*, beginning January 1st, cuts the mass-curve a little after September 1st. With 55 lockages per day, the lake would begin to drop about January 1st, reach

its minimum elevation about May 1st, would then begin to fill, coming to high-water mark during the first part of September, and then wasting until the beginning of the next dry season. From the right-hand diagram it is seen that in 1908 similar conditions would have been experienced with the same number of lockages per day, but, on account of heavier rainfall, the reservoir would have become full about 2 weeks earlier. Lines drawn for greater or less rates show correspondingly greater or lesser storage, depletion, and a similar condition for the reservoir to be below high water. The conditions referred to the left-hand diagram of Fig. 55 are shown in different form in Table 54.

In 1902-03, with a rate of 55 lockages per day, the deficiency in storage, which is the sum of the figures in Column 5, would have been 10.88 in., or 0.74 in. less than the requirement, which indicates that this rate could have been supplied during this season. The reservoir began to fill in June, and, up to September 1st, 10.47 in. over the whole land area had run into the lake, indicating that full reservoir would have occurred some time during the first part of September.

In 1907-08, with a rate of 55 lockages per day, the deficiency in storage, which is the sum of the figures in Column 10, would have been 11.60 in., or 0.02 in. less than the requirement. This is also just inside the limit (11.624). Beginning with May, just following the depletion, and summing up the monthly surplus in Column 9, it appears that the reservoir would have become full some time in August. The results given in these two cases check with those taken from the curve, as they should, as this method is only a rather laborious way of obtaining what can be much more easily taken directly from the mass-curve.

A study of the mass-diagram, Plate VII, shows that in the past decade, 1908 has not only been the year of least run-off, but is located at the bottom of the trough, the yield gradually decreasing from 1901 and rapidly increasing since May 1st, 1908. With a lake storage of a little more than 12 ft., or a limiting minimum lake elevation of + 75 instead of + 80.33, even during this period of low run-off, the lake would have been capable of caring for about 100 lockages per day. It appears, therefore, that under the present conditions of high-water elevation and storage it is possible to take care of 100 complete lockages per day without drawing the lake surface below Elevation + 75. Even with this rate of draft, the reservoir would have



MASS-CURVE
FOR
COMPUTATION OF AVAILABLE
YIELD AND STORAGE OF GATUN LAKE
TO MAINTAIN VARIOUS RATES OF LOCKAGE
IN PERIODS OF LEAST YIELD.

FIG. 55.

TABLE 54.—RELATION OF SUPPLY TO DRAFT—MINIMUM YEAR OF RUN-OFF.

Month, 1902-1903.	Yield, per square mile of land area, in inches.	55 LOCKAGES PER DAY. Require- ment, in inches.	Surplus (waste), in inches.	Deficiency (storage), in inches.	Month, 1907-1908.	Yield per square mile of land area, in inches.	55 LOCKAGES PER DAY. Require- ment, in inches.	Surplus (waste), in inches.	Deficiency (storage), in inches.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
May	4.19	2.85	1.34	....	May	1.58	2.85	....	1.27
June	4.12	2.75	1.37	....	June	5.93	2.75	3.18	....
July	4.93	2.85	2.08	....	July	6.57	2.85	3.72	....
August	4.65	2.85	1.80	....	August	5.99	2.85	3.14	....
September	5.78	2.75	3.03	....	September	7.92	2.75	5.17	....
October	9.26	2.85	6.41	....	October	13.15	2.85	10.30	....
November	9.99	2.75	7.24	....	November	8.72	2.75	5.97	....
December	4.08	2.85	1.23	....	December	4.49	2.85	1.64	....
January	1.11	2.85	....	1.74	January	0.62	2.85	....	2.23
February	0.07	2.58	....	2.51	February	—0.31	2.67	....	2.98
March	—0.54	2.85	....	3.39	March	—0.56	2.85	....	3.41
April	—0.31	2.75	....	3.06	April	—0.23	2.75	....	2.98
May	2.67	2.85	....	0.18	May	5.75	2.85	2.90	....
June	3.77	2.75	1.02	....	June	5.01	2.75	2.26	....
July	6.68	2.85	3.83	....	July	5.29	2.85	2.44	....
August	8.47	2.85	5.62	....	August	9.21	2.85	6.36	....
September	8.21	2.75	5.56	....	September	7.56	2.75	4.81	....
October	13.84	2.85	10.99	....	October	8.34	2.85	5.49	....
November	16.19	2.75	13.44	....	November	13.65	2.75	10.90	....

Requirement for different periods and different rates of lockage.

For 55 lockages per day, requirement, 2.476 cu. ft. per sec. per sq. mile.

Period, in days.		Requirement, in inches.
28	1.041 × 2.476	2.577
29	1.078 × 2.476	2.667
30	1.115 × 2.476	2.761
31	1.152 × 2.476	2.852

For 50 lockages per day. Requirement, 2.251 cu. ft. per sec. per sq. mile.

Period, in days.		Requirement, in inches.
28	1.041 × 2.251	2.34
29	1.078 × 2.251	2.43
30	1.115 × 2.251	2.51
31	1.152 × 2.251	2.59

NOTE.—The yield in Columns 2 and 7 was obtained in the same way as the figures in Column 8 of Table 52. Some of the figures there given are repeated in this table.

TABLE 55.—COMPARISON OF RAINFALL FOR 1910-11 WITH NORMAL RAINFALL FOR YEARS OF RECORD.

Year 1910-11.			Normal for Years of Record.		
Month.	Monthly, in inches.	Accumulated, in inches.	No. of Years.	Monthly, in inches.	Accumulated, in inches.
Cristobal Rainfall.					
November...	30.04	30.04	40	22.01	22.01
December...	15.20	45.24	40	12.58	34.59
January.....	0.99	46.23	41	4.05	38.64
February....	1.81	48.04	41	1.46	40.10
March........	1.41	49.45	42	1.69	41.79
April........	3.06	52.51	41	4.16	45.95
May.........	17.13	69.64	41	12.42	58.37
June........	15.58	85.22	41	13.29	71.66
July.........	14.58	99.80	41	16.46	88.12
August......	11.60	111.40	41	15.13	103.25
Gatun Rainfall.					
November...	26.06	26.06	6	24.03	24.03
December...	19.24	45.30	6	15.83	39.86
January.....	1.20	46.50	7	4.24	44.10
February....	2.19	48.69	7	2.41	46.51
March.......	1.80	50.49	7	3.25	49.76
April........	6.38	56.87	7	4.03	53.79
May.........	19.14	76.01	7	15.93	69.72
June.........	14.72	90.73	6	13.61	83.33
July..........	6.66	97.39	6	12.51	95.84
August.......	7.91	105.30	6	14.73	110.57
Bohio Rainfall.					
November...	22.37	22.37	17	19.39	19.39
December....	21.30	43.67	16	11.25	30.64
January.....	0.32	43.99	16	6.00	36.64
February....	1.87	45.86	16	2.11	38.75
March.......	0.97	46.83	16	2.01	40.76
April........	4.87	51.70	17	5.78	46.54
May..........	12.32	64.02	17	13.84	60.38
June.........	6.96	70.98	17	12.14	72.52
July.........	7.29	78.27	17	13.01	85.53
August......	5.69	83.96	17	14.77	100.30
Gamboa Rainfall.					
November...	16.90	16.90	28	12.54	12.54
December...	13.11	30.01	28	7.31	19.85
January.....	0.11	30.12	26	1.91	21.76
February....	0.71	30.83	26	0.84	22.60
March........	0.38	31.21	27	0.86	23.46
April........	4.01	35.22	30	3.63	27.09
May..........	14.53	49.75	30	11.01	38.10
June........	6.98	56.73	30	9.63	47.73
July..........	7.26	63.99	31	10.32	58.05
August.......	7.68	71.67	29	12.04	70.09

TABLE 55.—(*Continued.*)

Month.	Year 1910-11. Monthly, in inches.	Year 1910-11. Accumulated, in inches.	Normal for Years of Record. No. of Years.	Normal for Years of Record. Monthly, in inches.	Normal for Years of Record. Accumulated, in inches.
Alhajuela Rainfall.					
November...	14.87	14.87	12	14.70	14.70
December ...	15.49	30.36	12	8.04	22.74
January	0.14	30.50	12	1.33	24.07
February....	2.29	32.79	12	0.83	24.90
March	0.01	32.80	12	0.71	25.61
April.........	4.88	37.68	12	3.47	29.08
May..........	16.11	53.79	11	12.69	41.77
June.........	10.53	64.32	12	12.92	54.69
July	8.84	73.16	13	13.47	68.16
August.......	10.79	83.95	13	13.10	81.26
Culebra Rainfall.					
November...	10.81	10.81	22	12.52	12.52
December ...	11.86	22.67	20	8.19	20.71
January	0.02	22.69	21	1.83	22.54
February ...	0.74	23.43	21	0.57	23.11
March........	0.06	23.49	21	0.71	23.82
April	4.89	28.38	21	3.83	27.65
May	14.86	43.24	21	11.31	38.96
June	4.25	47.49	20	8.92	47.88
July	5.95	53.44	20	9.50	57.38
August......	8.36	61.80	20	10.46	67.84
Trinidad Rainfall.					
November...	22.95	22.95	3	23.47	23.47
December...	21.64	44.59	3	15.79	39.26
January.....	0.64	45.23	4	3.80	43.06
February....	2.00	47.23	4	3.28	46.34
March........	2.33	49.56	4	3.89	50.23
April	3.33	52.89	4	5.74	55.97
May	18.20	71.09	4	14.65	70.62
June	8.79	79.88	4	10.90	81.52
July.........	3.63	83.51	4	8.94	90.46
August	8.17	91.68	4	12.18	102.64
Monte Lirio Rainfall.					
November...	25.60	25.60	3	28.63	28.63
December ...	23.96	49.56	4	14.74	43.37
January.....	0.89	50.45	4	3.98	47.35
February....	2.41	52.86	4	4.60	51.95
March	1.41	54.27	4	4.64	56.59
April	5.27	59.54	4	5.77	62.36
May	19.70	79.24	4	15.84	78.20
June.........	11.43	90.67	4	13.64	91.84
July	10.58	101.25	4	14.76	106.60
August.	9.58	110.83	4	11.88	118.48
Porto Bello Rainfall.					
November...	23.08	23.08	4	31.85	31.85
December....	22.89	45.97	4	30.07	61.92
January.....	0.84	46.81	4	9.64	71.56
February....	6.90	53.71	4	5.28	76.84
March.......	2.38	56.09	4	3.30	80.14
April	6.60	62.69	4	8.07	88.21
May	19.25	81.94	4	14.82	103.03
June.........	18.04	99.98	4	17.00	120.03
July	14.48	114.46	4	18.40	138.43
August......	22.56	137.02	4	18.52	156.95

filled by January 1st, 1909. The general trend of the mass-curve indicates that the rate of 100 lockages per day is the reasonable maximum capacity of the canal, as far as water supply is concerned. Below this rate the storage lies in the present lake, provided Elevation + 75 can be allowed for the minimum surface elevation. For more than 100 lockages per day, the storage requirement increases so rapidly that it could be developed only at a considerable expense.

With practically unlimited storage during the past decade, the run-off would have been sufficient to have provided for about 125 lockages per day. To do this would probably mean additional storage reservoirs on tributary streams to conserve water which, under present conditions, will waste during the rainy months.

The rainfall on the Isthmus since January 1st, 1911, has been light, and below normal at many stations. The run-off for the tributary area has been affected, and the monthly flow past Gatun is the lowest of record (either by measurement or estimation) since 1890. Comparisons of the rainfall records in the drainage area for this period with the normals for the years of record are given in Table 55.

Table 56 is a comparison of the run-off at Alhajuela and Gatun for the present period, and the average of the monthly records of these stations for 1890-1911.

TABLE 56.

Month.	Alhajuela Station. Mean run-off, in inches.				Gatun Station. Mean run-off, in inches.			
	1911.	Accumulated.	1890-1911, inclusive.	Accumulated.	1911.	Accumulated.	1890-1911. inclusive	Accumulated.
January...	4.427	4.427	6.214	6.214	3.170	3.179	5.358	5.358
February..	5.634	10.061	3.126	9.340	2.364	5.534	2.096	7.454
March.....	2.375	12.436	2.079	11.419	1.257	6.791	1.417	8.871
April......	4.418	16.854	2.967	14.386	1.752	8.543	1.991	10.862
May........	7.528	24.382	6.122	20.508	5.200	13.743	5.396	16.258
June.......	7.531	31.913	6.216	26.721	5.489	19.232	5.627	21.885
July........	7.044	38.957	7.924	34.648	6.030	25.262	7.467	29.352
August....	6.364	45.320	8.157	42.805	3.954	29.216	8.181	37.533
September.	4.786	50.107	7.529	50.334	3.805	33.021	7.864	45.397

The run-off past Gatun is affected constantly by two conditions which do not obtain in the basin above Alhajuela:

1.—Extensive swamp areas with a cover of vegetation that require large quantities of water; and

2.—Evaporation from Gatun Lake surface, which was not experienced in former years.

In addition to the foregoing, the greatest factor contributing to the large proportional quantity of run-off per square mile above Alhajuela, compared with that of the basin as a whole, probably is to be found in the greater precipitation on the former area. This condition is found in Tables 44 and 45, from which it appears that the annual average rainfall from the whole basin for 10 years was 125.04 in.; for the portion below Alhajuela, 118.90 in.; and for the portion above that station, 136.36 in. In other words, the rainfall on the area above Alhajuela is about 9% greater than the average on the basin as a whole, and 14.5% greater than that on the basin below that station.

TABLE 57.

Month.	Mean lake elevation.	Mean area of lake surface, in square miles.	Evaporation, Brazos Brook, in inches.	Evaporation in cubic feet per second, per square mile.	Equivalent evaporation for actual lake surface, in cubic feet per second.	Equivalent evaporation for proposed lake area (167.4), in cubic feet per second.
	(1)	(2)	(3)	(4)	(5)	(6)
January...	14.09	19.60	6.29	5.46	107	913
February..	13.71	18.90	5.12	4.92	93	822
March. ...	12.55	17.30	6.87	5.96	103	998
April......	13.12	18.10	4.94	4.43	80	740
May.......	15.58	21.70	3.29	2.85	62	478
June.......	15.58	22.10	2.92	2.62	58	438
July.......	15.92	22.30	4.36	3.78	84	634
August....	14.50	20.50	4.07	3.53	72	592
September	14.48	20.45	4.10	3.56	73	596

The figures in Column 5 of Table 57 should be added to the monthly discharge of Table 28, since April, 1910, in order to compare these 3 months with the discharge of previous months of the same name.

Table 58 is given to show what would have been the effect on the lake and its usefulness had it been in operation during the period since January 1st, 1911.

Table 58 shows that, even with the low rainfall and similar conditions of run-off since January 1st, 1911, the lake would have handled 55 lockages per day, and the surface then would have been above the limit by 2.38 ft., equivalent to 4.15 in. in land surface.

TABLE 58.—FIFTY-FIVE LOCKAGES PER DAY.

Month, 1910-11.	Yield, corrected for evaporation on lake area, in inches.	Requirement, in inches.	Surplus.	Deficiency.
November	16.98	2.75	14.23	......
December	21.01	2.85	18.16	......
January	1.80	2.85	......	1.05
February	1.38	2.58	......	1.20
March	0.23	2.85	......	2.62
April	0.15	2.75	......	2.60
May	6.51	2.85	3.66	......
June	5.92	2.75	3.17	......
July	5.75	2.85	2.90	......
August	3.90	2.85	1.05	......
September	3.62	2.75	0.87	......

From that which has preceded, therefore, it appears that the water supply of Gatun Lake is sufficient for any probable demands of commercial or other navigation, and the use of the canal is limited only by the capacity of the locks and the requirements of practical operation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

CONSTRUCTION PROBLEMS, DUMBARTON BRIDGE, CENTRAL CALIFORNIA RAILWAY.

By E. J. Schneider, M. Am. Soc. C. E.
To be Presented March 19th, 1913.

It is not the object of this paper to give a complete description of all the construction features of the Dumbarton Bridge. The aim and purpose is to outline, in a general way, the field methods, and to discuss details only where exceptional problems were met.

To secure an all-rail route into the City of San Francisco from the main line tracks on the eastern shore of San Francisco Bay, thus to avoid ferrying increasing numbers of freight cars between Oakland and San Francisco, the Central California Railway, better known locally as the Dumbarton Cut-off (Fig. 1), was projected and constructed by the Southern Pacific Company during the three years ending September 30th, 1910. The cut-off connects Niles and Newark on the east side with Redwood City on the west side of the bay. Through this improvement, the all-rail distance between Oakland and San Francisco has been reduced 26.1 miles, as compared with the corresponding earlier distance *via* San José.

The bridge proper consists of a draw span, 310 ft. in length over all, and three 180-ft., double-track, through, riveted, approach spans at each end of the draw. All structural steelwork was designed in accordance with the Harriman Lines Common Standards of 1906.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

using what corresponds to a Cooper E-55 loading. The center to center distance of end pins is 1 474 ft.

There is also a considerable length of single-track, creosoted timber, ballasted trestle at each end of the steelwork. The length of the east approach trestle is 1 002 ft., and that of the west approach 5 366 ft. The total length of the entire structure, steel spans and timber trestles, is 7 842 ft., or approximately 1½ miles.

No unusual methods were required in building the timber trestles, which are all of standard type. The bents are 15 ft. on centers, and each bent consists of six piles. The piles are of Oregon fir, and in lengths ranging from 60 to 120 ft., the penetrations in some cases being as great as 60 ft., depending on the depth of the mud.

The specifications call for piles of 10-in. tip in the pier foundations, and for a 9-in. tip for those used in the trestles. The butt diameters were limited only by the width of the gin, which was 22 in. The trestle piles were creosoted; those for the foundation piers supporting the structural steel spans were not creosoted. Fig. 1, Plate VIII, shows the pile-drivers used.

Test piles, driven at about 300-ft. centers along the proposed bridge axis, indicated that the depth of the mud varied from 2 to 4 ft. near the middle of the channel, and from 16 to 18 ft. toward the shores of the bay. Under the mud was found a stratum of fine black sand, mixed with gravel, averaging from 15 to 20 ft. in depth, and below this a hard clay. It is interesting to note that no trace of clay was found by the Spring Valley Water Company when making tests only about 500 ft. to the north and on a line paralleling the Dumbarton structure.

According to the original plans, it was the intention to extend the wooden trestles from each shore for the full approach lengths to the rest piers of the draw span; but when the east approach had been completed to within about 120 ft. of the draw, on August 21st, 1907, at 3 A. M., a length of about 120 ft. of trestle was undermined and washed out, precipitating the pile-driver equipment into the bay. There was no storm at the time; the accident was due entirely to the currents produced by the outgoing high tide.

These difficulties at once prompted a modification of the original trestle plans. It was first proposed to alter 200 ft. of the east approach and 600 ft. of the west approach, immediately adjoining the

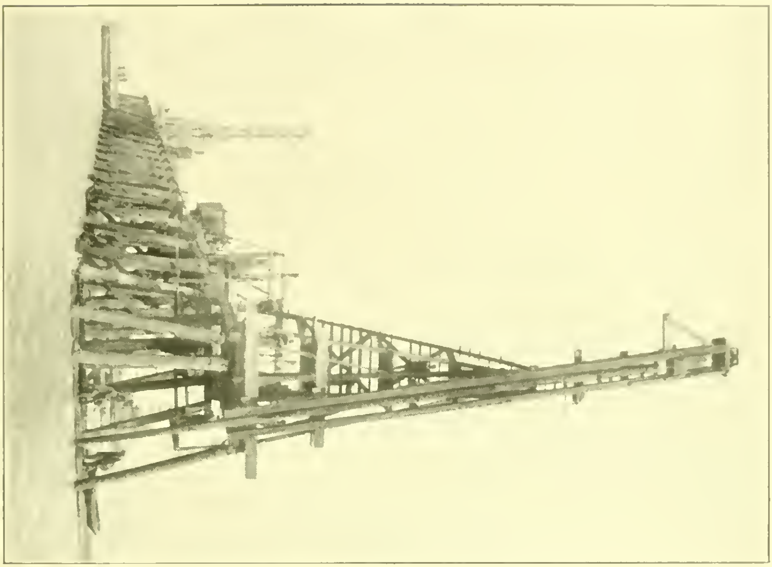


Fig. 1.—Driving Piles for Dumbarton Point Trestle.

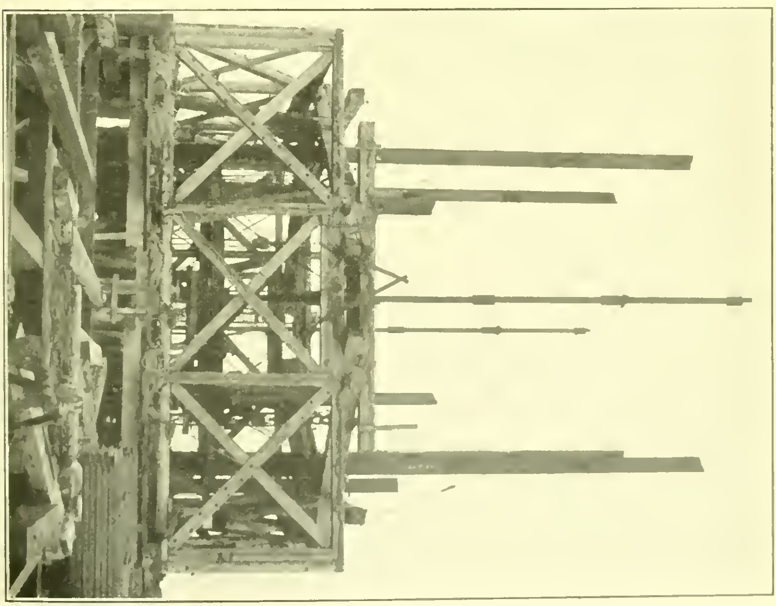


Fig. 2.—Erection of Guide-Frame for Cylinder Pier, Dumbarton Point Draw.

Fig. 1.—Erection of Falsework for Pivot Pier, Dumbarton Point Draw.

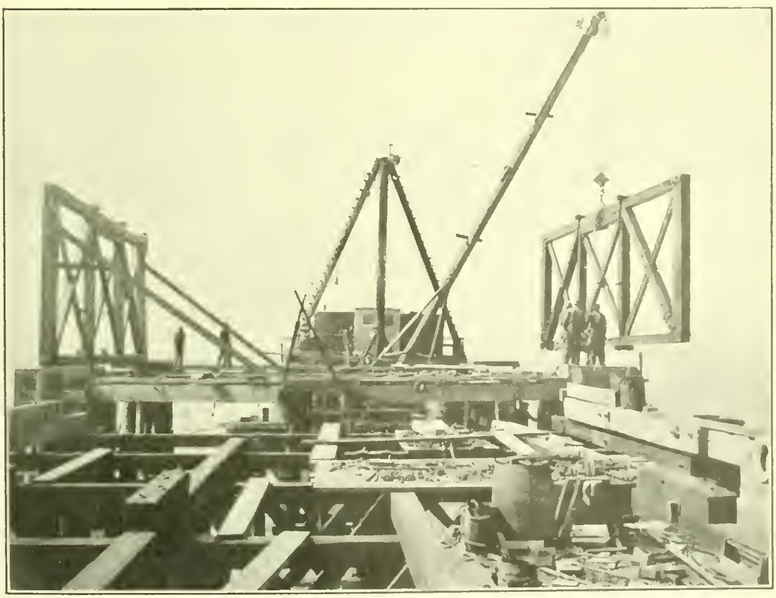


Fig. 2.—Erection of Falsework for Pivot Pier, Dumbarton Point Draw.

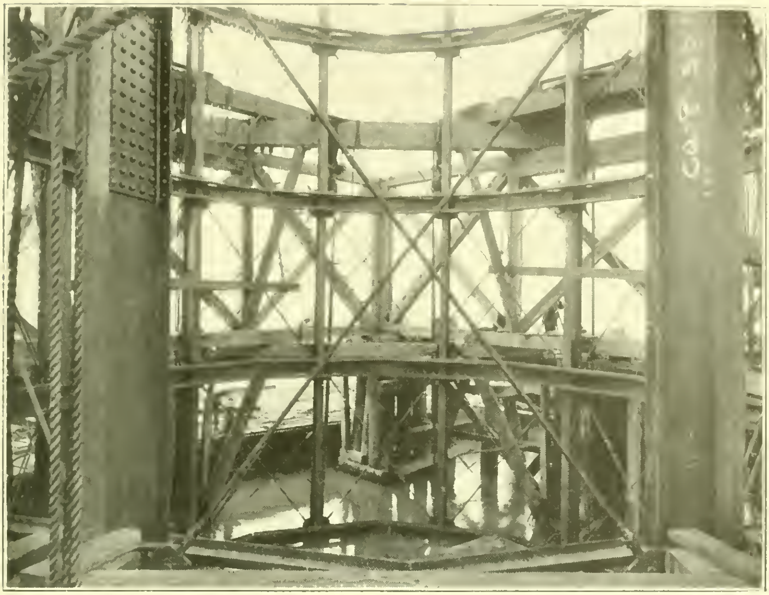


FIG. 1.—INTERIOR OF GUIDE-FRAME FOR PIVOT PIER CASING, DUMBARTON POINT DRAW.

FIG. 2.—THREE LOWER COURSES OF STEEL SHELL FOR CENTER PIER, DUMBARTON POINT DRAW.

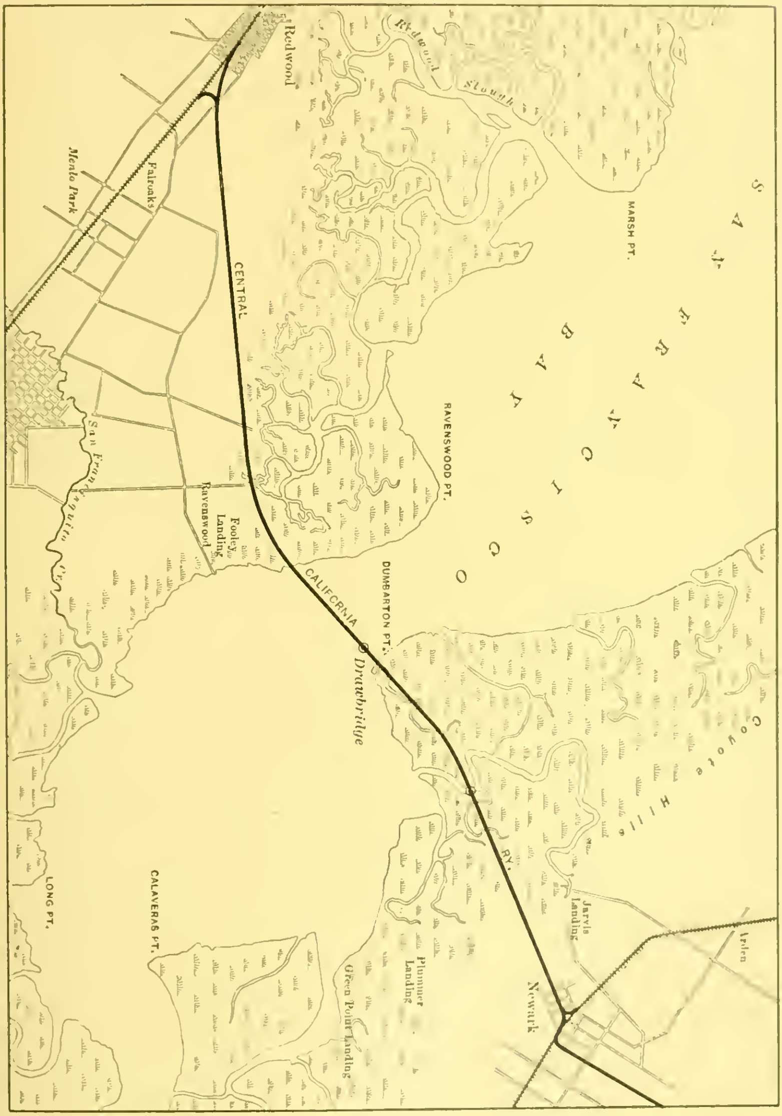

Redwood
Redwood Slough
Menlo Park
Fairoaks
CENTRAL
CALIFORNIA
RY.
SAN FRANCISCO BAY
MARSH PT.
RAVENSWOOD PT.
DUMBARTON PT.
Drawbridge
Ravenswood
Fooley Landing
San Francisquito Cr.
LONG PT.
CALAVERAS PT.
Green Point Landing
Plummer Landing
Newark
Jarvis Landing
Coyote Hills

FIG. 1.

draw span, by supporting the track for these distances on eleven double-track pile bents. After a few of these bents had been driven, it was seen that, by this method, the trestle could not be built steady enough to prevent vibration while high tides were running out. Thus a second modification was decided on, substituting for portions of the trestle approaches, six 180-ft. steel spans, three on each side of the draw (Fig. 2).

At this location in the bay, unusual and difficult foundation problems presented themselves, due to depth of water, the tides, and the nature of the soil. The depth of water at the pivot pier at mean low tide is 51 ft.; at extreme high tide it is 58.3 ft., with an extreme range of tide levels of 11.7 ft. Very rapid currents were caused by the large prisms of water required to pass through the narrow channel, approximately 1 500 ft. wide. Measurements registered a maximum flow of 4½ miles per hour.

These conditions finally led the engineers to adopt cylinder foundations to support the steel superstructure, and these have proved entirely satisfactory. They selected for the type of pier a cylindrical steel shell, enclosing piles and concrete. A description of the 40-ft. pivot pier will illustrate the type, which was also used for all smaller piers. Eight clusters of piles were driven to form a hollow square, 48 ft. on a side, with the center of the square at the center of the pier. About 10 ft. of mud and sand were then dredged, using an orange-peel bucket. The dredged material was placed in a bottom-dropping scow, transported, and dumped into the bay a short distance from the bridge site. Falsework (Figs. 1 and 2, Plate IX), consisting of four wooden trusses, 44 ft. long and 14 ft. high, was constructed of 12 by 12-in. timbers and supported on the pile clusters, for the purpose of sustaining and erecting the guide-frame (Fig. 2, Plate VIII) for the steel cylinders. The guide-frame consisted of twelve 20-in., 65-lb. I-beams, 80 ft. long, placed upright to form a regular, twelve-sided polygon about 39 ft. in diameter. These I-beam columns were braced horizontally at intervals of 12 ft. 6 in. by two 12-in. channels placed back to back, as shown by Fig. 1, Plate X. Additional stiffening was obtained by reinforcing the top and bottom of these 12-in. channels at their mid-points with 8-in. channels. Thus was formed a series of trusses (Fig. 3) which added greatly to the lateral stiffness of the entire frame. In general appearance the guide-

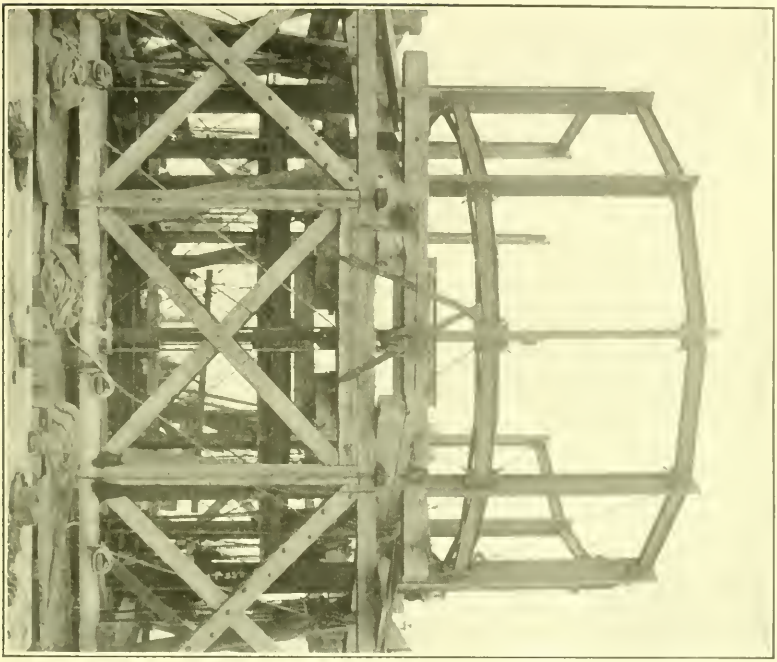


FIG. 1.—ERECTION OF GUIDE-FRAME FOR PIVOT PIER CASING, DUMBARTON POINT DRAW.

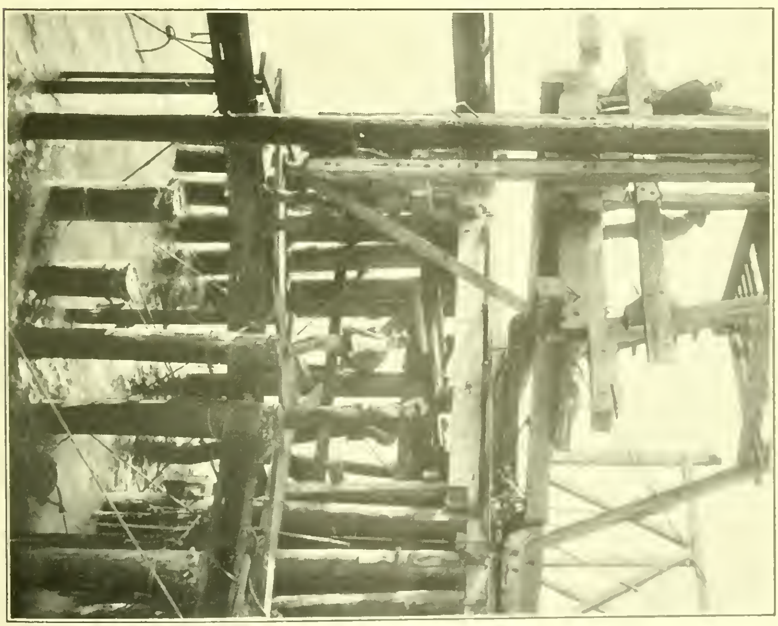


FIG. 2.—DRIVING FOUNDATION PILES FOR PIVOT PIER, DUMBARTON POINT DRAW.

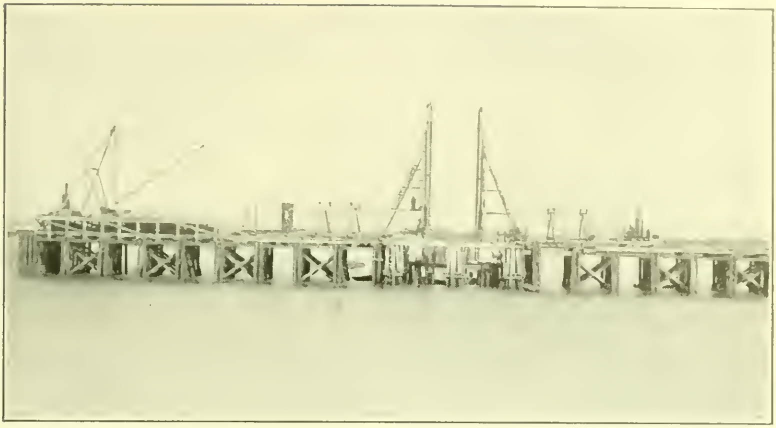


FIG. 1.—DRIVING FOUNDATION PILES FOR PIVOT PIER, DUMBARTON POINT DRAW.

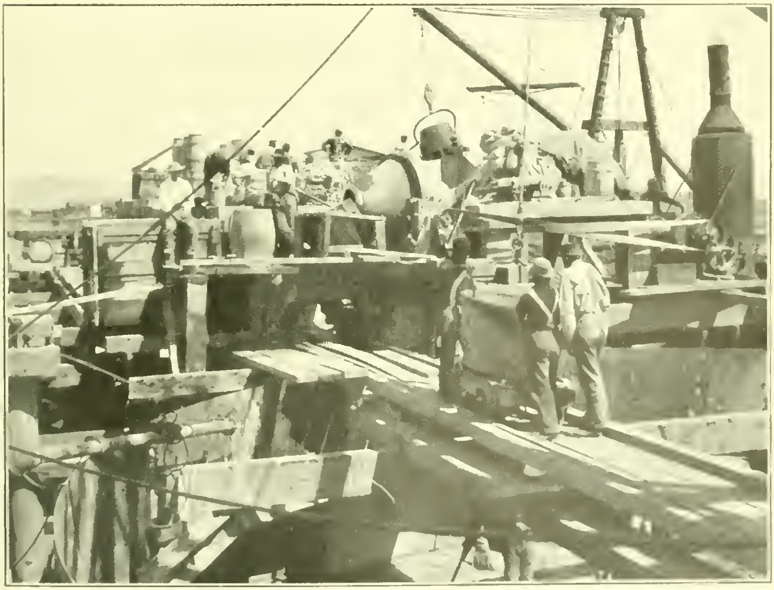


FIG. 2.—PLACING CONCRETE IN PIVOT PIER, DUMBARTON POINT DRAW

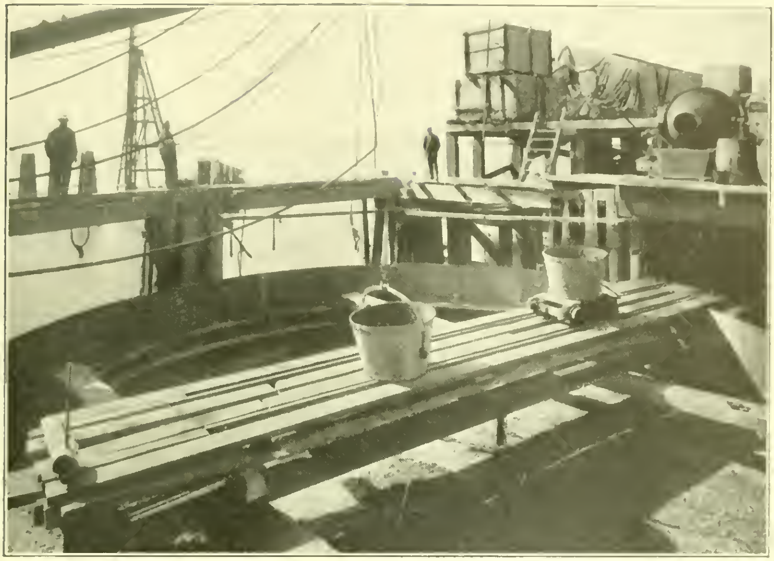


Fig. 1.—Concrete Plant, Pivot Pier, Dumbarton Point Draw.

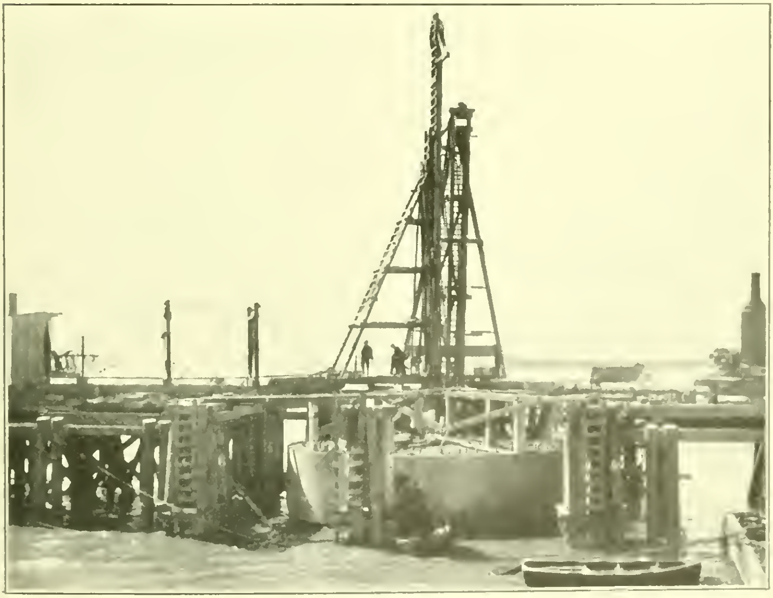


Fig. 2.—Pulling Falsework Dolphin Piles, Dumbarton Point Draw.

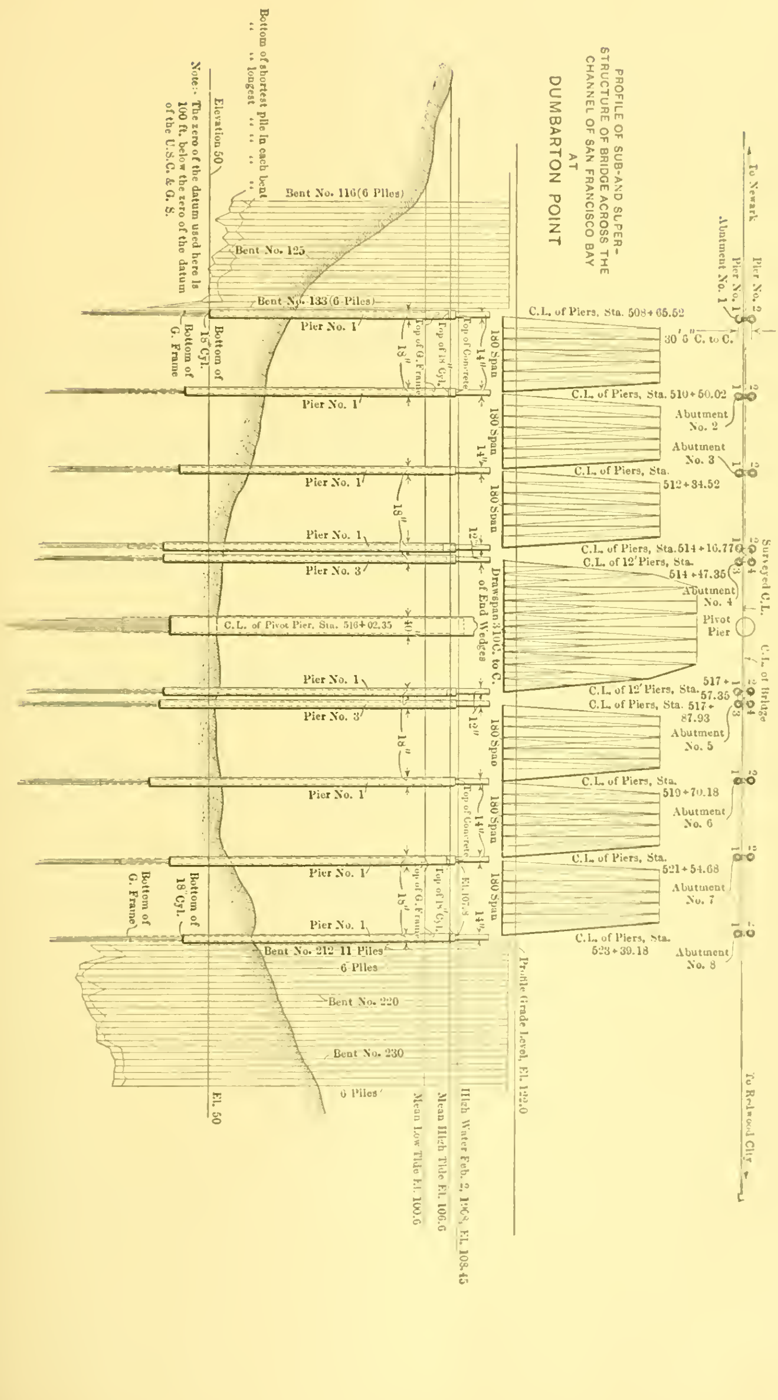

PROFILE OF SUB-AND SUPER-STRUCTURE OF BRIDGE ACROSS THE CHANNEL OF SAN FRANCISCO BAY AT
DUMBARTON POINT
To Newark
To Redwood City
Bottom of shortest pile in each bent
" " longest " " " "
Note:- The zero of the datum used here is 100 ft. below the zero of the datum of the U.S.C. & G. S.
Elevation 50
El. 50
Bent No. 116 (6 Piles)
Bent No. 125
Bent No. 133 (6 Piles)
Bent No. 212 11 Piles
6 Piles
Bent No. 220
Bent No. 230
6 Piles
Pier No. 1
Pier No. 3
C.L. of Pivot Pier, Sta. 516+02.35
Bottom of 18" Cyl.
Bottom of G. Frame
Top of G. Frame
Top of 18" Cyl.
Top of Concrete
180 Span
Drawspan 310 C. to C. of End Wedges
C.L. of Piers, Sta. 508+65.52
30' 6" C. to C.
C.L. of Piers, Sta. 510+50.02
C.L. of Piers, Sta. 512+34.52
C.L. of Piers, Sta. 514+16.77
C.L. of 12" Piers, Sta. 514+47.35
C.L. of 12" Piers, Sta. 517+57.35
C.L. of Piers, Sta. 517+87.93
C.L. of Piers, Sta. 519+70.18
C.L. of Piers, Sta. 521+54.68
C.L. of Piers, Sta. 523+39.18
Abutment No. 1
Abutment No. 2
Abutment No. 3
Abutment No. 4
Abutment No. 5
Abutment No. 6
Abutment No. 7
Abutment No. 8
Pivot Pier
Surveyed C.L.
C.L. of Bridge
Profile Grade Level, El. 122.0
High Water Feb. 2, 1908, El. 108.45
Mean High Tide El. 106.6
Mean Low Tide El. 100.6

FIG. 2.

frame resembled the framework for a gas holder (Fig. 1, Plate XI). As the frame was built it was lowered by tackle until it rested on the soil. Then it was weighted to refusal by loading 300 tons of steel on its top. To assist in securing an even settlement, a $\frac{3}{8}$-in. water jet was used, while the I-beam columns were tapped with a pile hammer. The guide-frame came to refusal when the first set of horizontal channel struts rested on the sand bottom. These struts were 17 ft. 4 in. above the lowest parts of the frame. The three lower courses (Fig. 2, Plate X) of the steel cylinder were then assembled on the falsework, and with rope tackles were lowered as a unit into position. A clearance of $\frac{1}{2}$ in. was allowed between the outer flange of the upright I-beams and the webs of the guide-channels which were attached to the shell. A rail was fastened to the top of the section of the shell with clips at each of the twelve points adjacent to the upright I-beam columns of the guide-frame. These rails were used as level rods, being appropriately marked. The shell was driven to refusal, with an even settlement, the bottom edge resting 6 ft. below the lowest horizontal struts of the guide-frame.

The space within the cylinder shell was then further excavated, the center portion being dredged about 5 ft. deeper than at the outside, which just about offset the uplift of the ground, due to the driving of the outside row of piles first and working toward the center with succeeding piles. Two skid pile-drivers, with an overhang of 30 ft., were used (Fig. 1, Plate XII), working from the pier protection used to support them. Inside the steel cylinder 141 piles were driven, at about 32-in. centers, as shown by Fig. 3. Care was taken to place each pile as nearly as possible to match its position on the pile plan. A pile, in position for driving, is shown by Fig. 2, Plate XI. A diver cut off two-thirds of the piles at an average height of 3 ft. above the bottom of the cylinder; the remaining piles extended to within 15 ft. of the top of the pier, or about 13 ft. below the top of the guide-frame.

Before the second unit of the steel cylinder was placed, the lower section was filled with concrete to within 7 ft. of its top, this being the highest level to which the concrete could be placed without disturbance from the tidal currents passing over the top of the section. A bottom-dump bucket was used, having a capacity of 18 cu. ft. Care was taken to provide an even distribution of concrete. A diver was

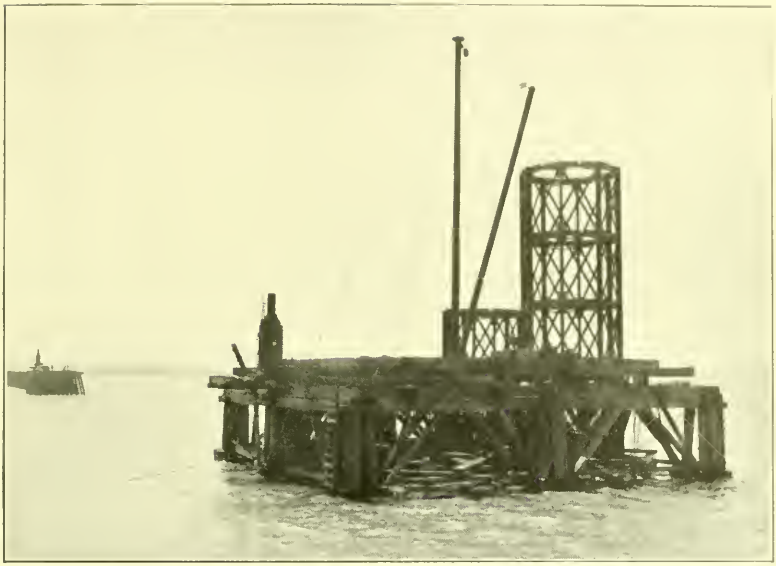


FIG. 1.—ERECTION OF GUIDE-FRAMES, WEST ABUTMENT PIER, DUMBARTON POINT DRAW.

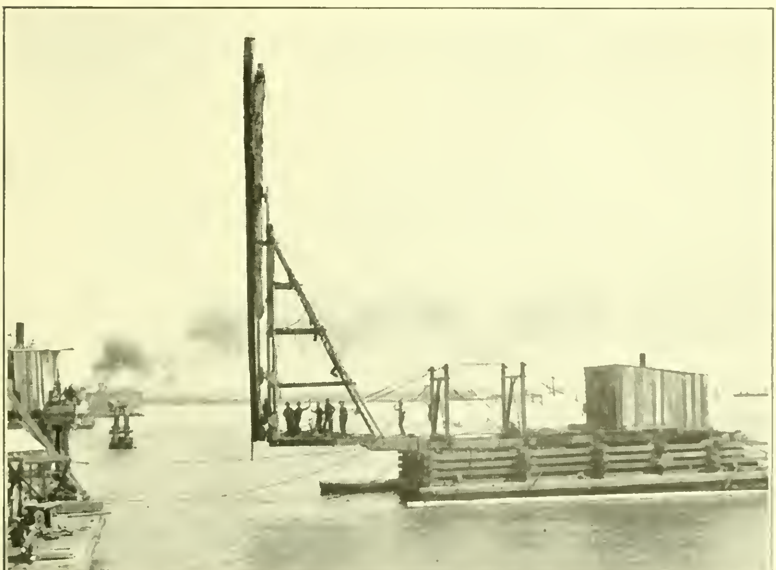


FIG. 2.—SKID PILE-DRIVER ON SCOW, WITH PILE CUT-OFF IN GINS, EAST ABUTMENT, DUMBARTON POINT DRAW.

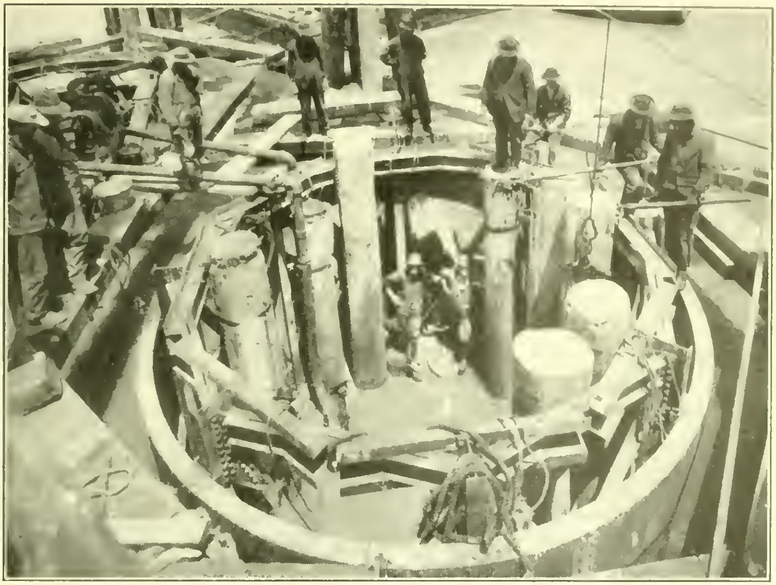


FIG. 1.—CUTTING OFF PILES IN PIER 2, ABUTMENT 4, DUMBARTON POINT.

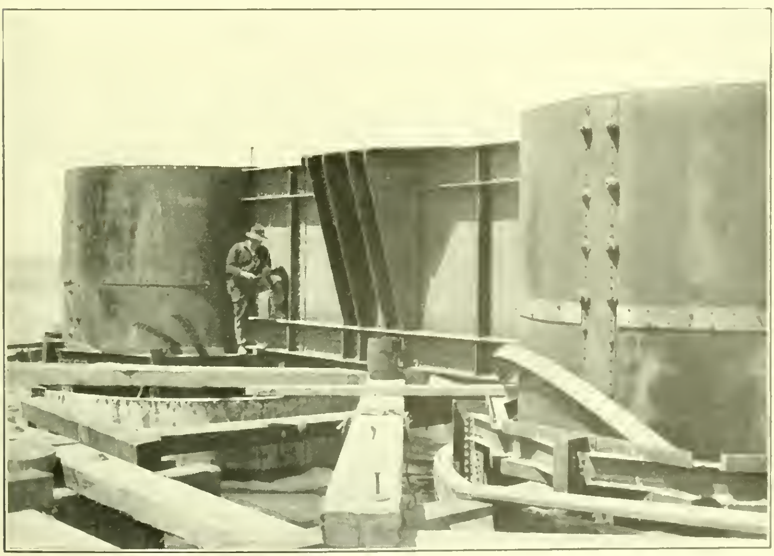


FIG. 2.—CYLINDERS IN PLACE, ABUTMENT 4, DUMBARTON POINT.

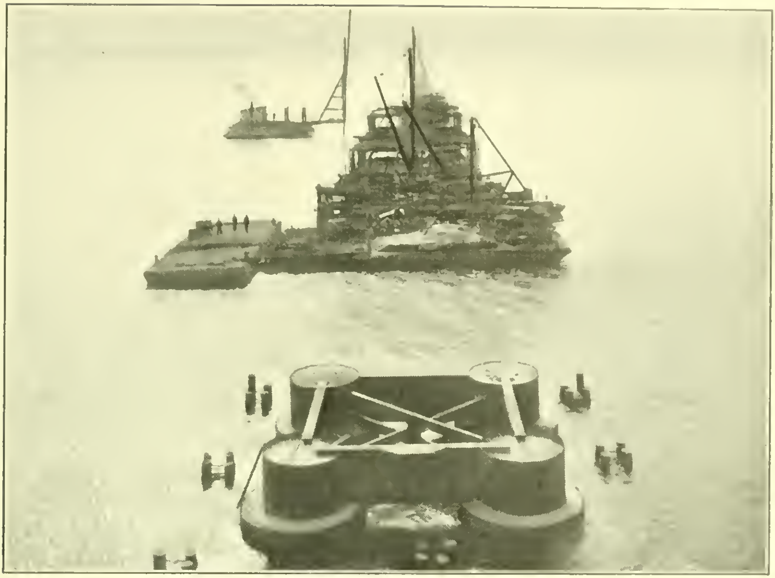


FIG. 1.—LOOKING WEST FROM ENGINE-HOUSE, ON DUMBARTON POINT DRAWSPAN.

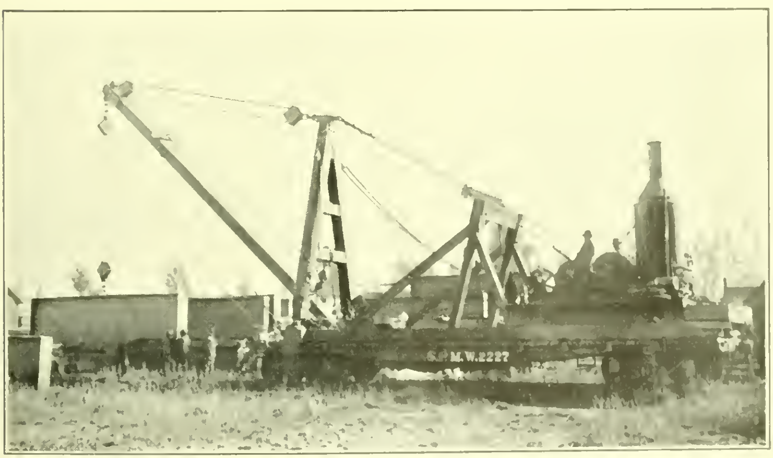


FIG. 2.—DERRICK CAR, HANDLING BRIDGE MATERIAL.

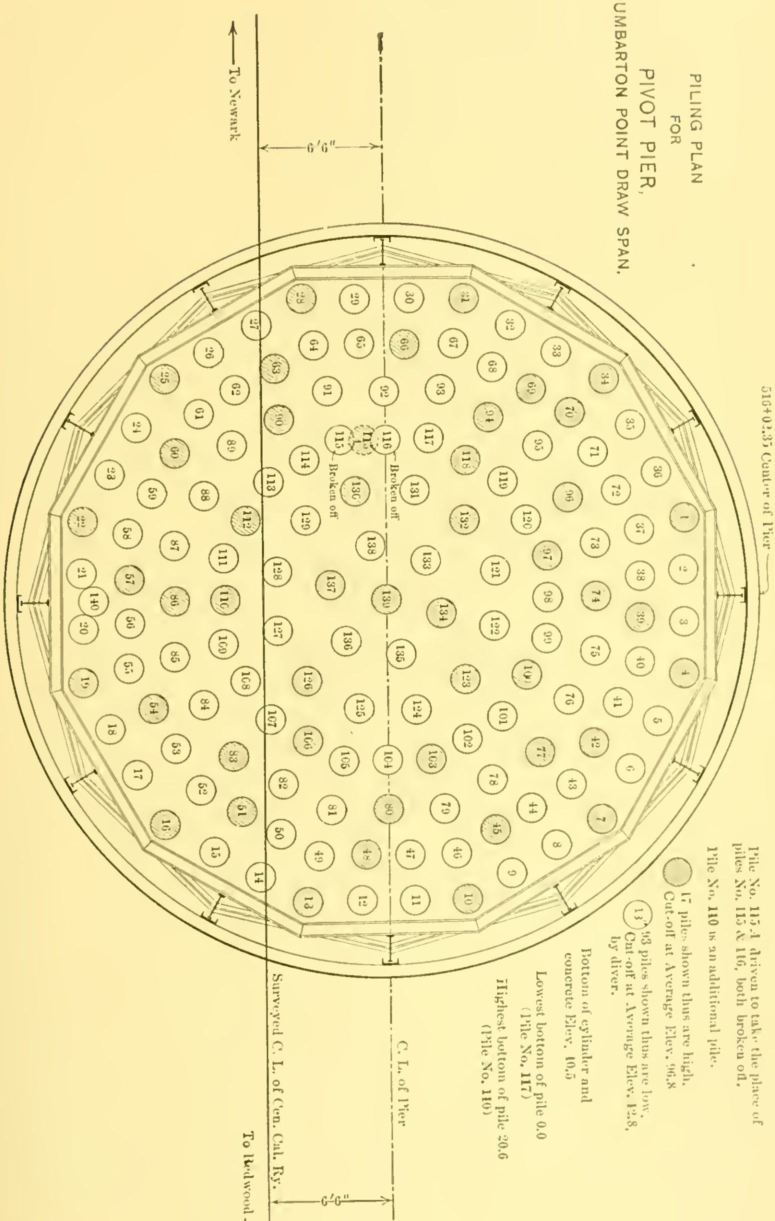

PILING PLAN
FOR
PIVOT PIER,
DUMBARTON POINT DRAW SPAN.
516+02.37 Center of Pier
Pile No. 115 A driven to take the place of piles No. 115 & 116, both broken off.
Pile No. 140 is an additional pile.
17 piles shown thus are high.
Cut-off at Average Elev. 96.8
93 piles shown thus are low.
Cut-off at Average Elev. 12.8, by diver.
Bottom of cylinder and concrete Elev. 10.5
Lowest bottom of pile 0.0 (Pile No. 117)
Highest bottom of pile 20.6 (Pile No. 140)
Broken off
Broken off
6′6″
6′6″
To Newark
To Redwood
C. L. of Pier
Surveyed C. L. of Cen. Cal. Ry.

FIG. 3.

employed continuously. He found that the concrete materials had very little tendency to spread, because the concrete was being deposited in still water. The second unit of the shell was then assembled and lowered to place.

To provide for a good connection between the units, a rather unique detail (Fig. 4) was evolved. The guide-frame was relied on to bring the joints to proper position and hold them there, but, to insure a more perfect bearing, continuous, flanged-steel plates, ½ in. thick, were

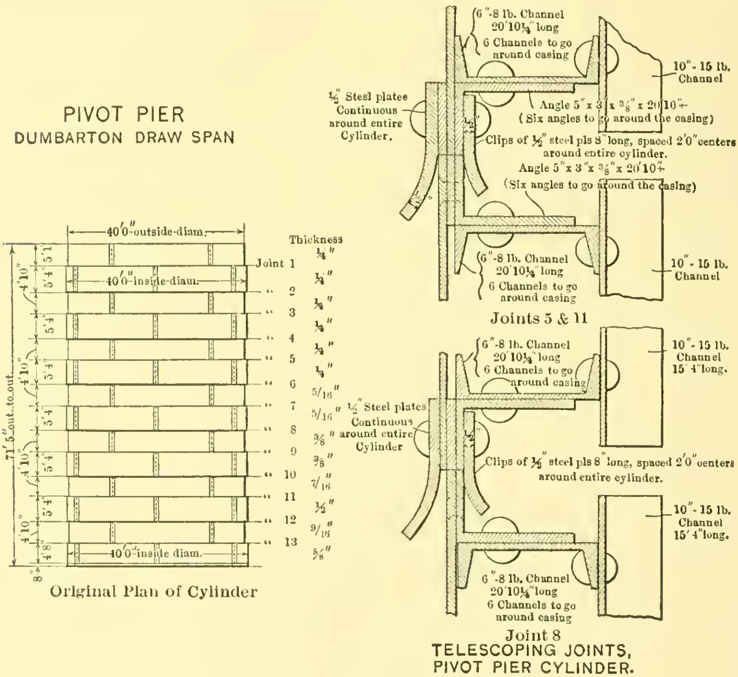


FIG. 4.

riveted around the entire outside of the cylinder on the upper sections at Joints 5, 8, and 11. Flanged clips, of ½-in. steel plates 8 in. long, were placed around the interior of the cylinder, and were 2 ft. from center to center. Thus a butt joint was formed, with cylindrical flange plates on the outside and flange clips on the inside. This portion of the work was also watched very carefully by a diver as the unit was lowered into position, the result being that an even bearing was obtained and also a very satisfactory connection.

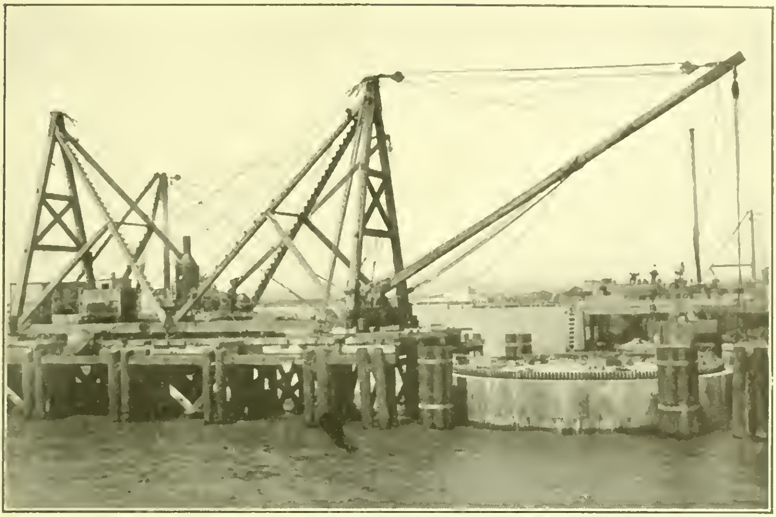


FIG. 1.—DOUBLE DERRICK TRAVELER FOR ERECTION OF DUMBARTON POINT DRAW.

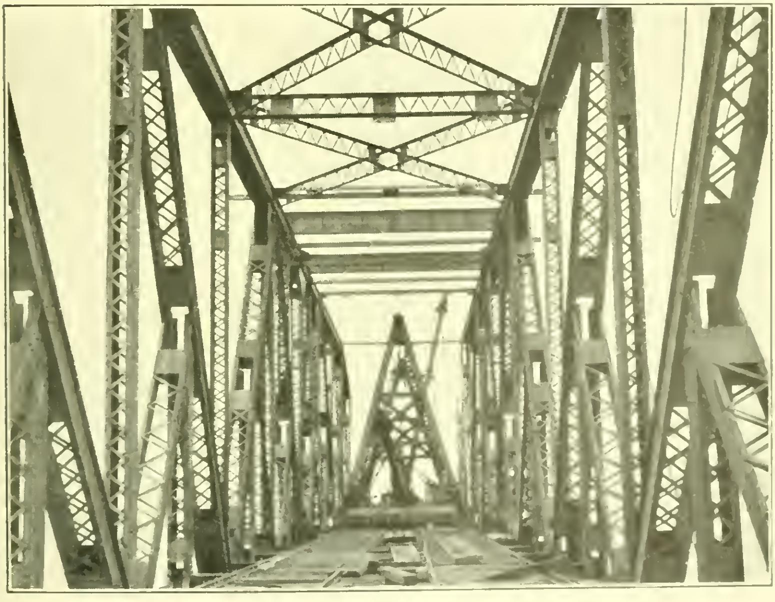


FIG. 2.—ERECTION OF DUMBARTON POINT DRAW.

FIG. 1.—ERECTION OF DUMBARTON POINT DRAW.

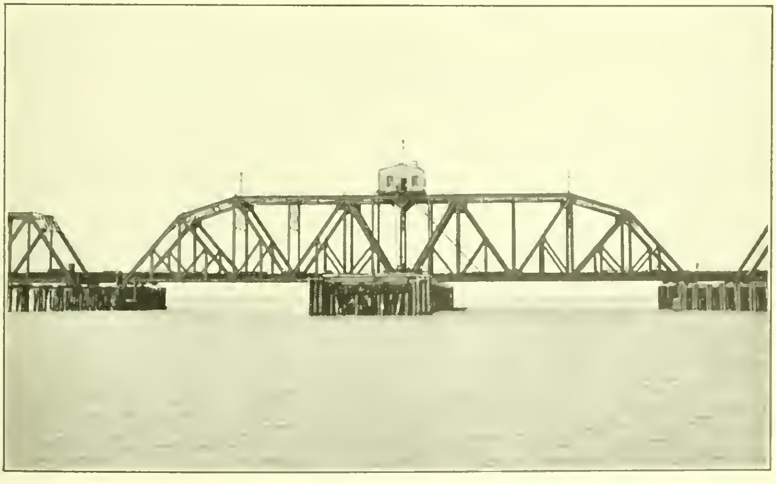


FIG. 2.—DUMBARTON POINT DRAW.

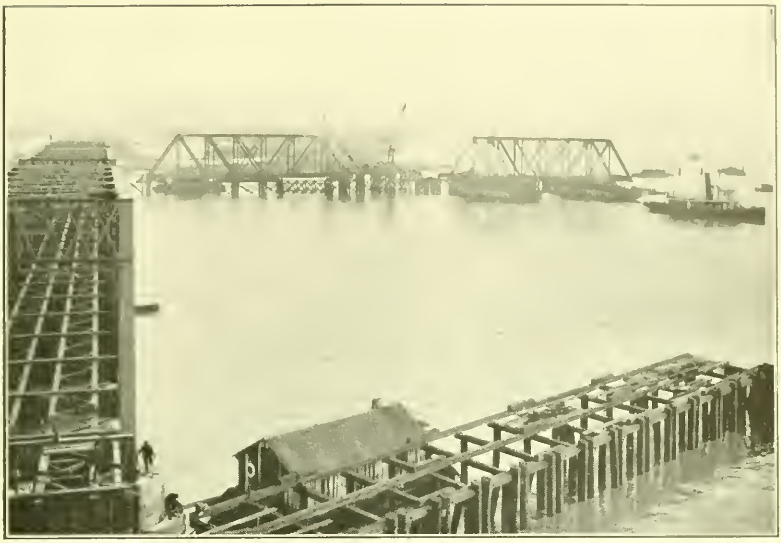
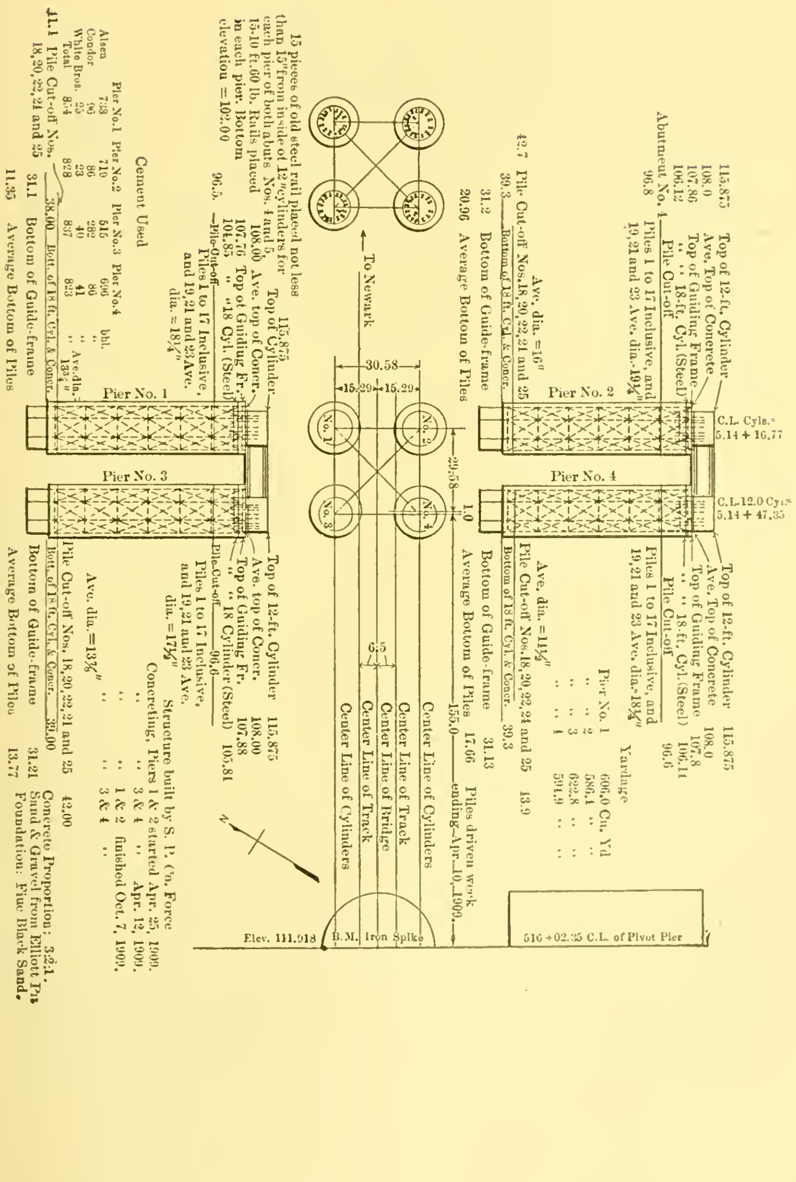


FIG. 1.—FLOATING SPAN 5 INTO POSITION, DUMBARTON POINT BRIDGE.

FIG. 2.—FLOATING SPAN 1 INTO POSITION, DUMBARTON POINT BRIDGE.



Pier No. 1
Pier No. 2
Pier No. 3
Pier No. 4
To Newark
Center Line of Cylinders
Center Line of Track
Center Line of Bridge
Center Line of Track
Center Line of Cylinders
Elev. 111.918
B.M. Iron Spike
516 + 02.35 C.L. of Pivot Pier
Cement Used
Yardage
Structure built by S. P. Co. Force
Foundation: Fine Black Sand.

FIG. 5.

Concrete was placed to a level of about 7 ft. from the top of the second unit in the same manner as already described. The various operations were continued until the pier was completed to a level above the permanent water line. After this stage of the work 1 000-lb. coal buckets were used to place the concrete, as shown by Fig. 2, Plate XII, and Fig. 1, Plate XIII.

The falsework trusses were dismantled and taken down when the last section of the steel cylinder had been lowered into place (Fig. 2, Plate XIII). The piles of the falsework dolphins were pulled with the aid of two donkey engines.

The construction for the end abutment piers of the draw span, and also for the piers of the approach spans (Fig. 1, Plate XIV), was carried on in approximately the same way as for the pivot pier, so far as work below water was concerned. As no protection was provided for the abutment piers, the foundation piles were driven with a skid driver resting on a scow. This driver (Fig. 2, Plate XIV) was elevated on cribwork to provide the necessary clearance above the guide-frames. In the smaller piers only 5 of the 25 piles were cut off at a low level by a diver (Fig. 1, Plate XV); the remaining 20 piles were allowed to extend well up into the top of the shell.

With the exception of the pivot pier, all foundation cylinders were 18 ft. in diameter to an elevation 2 ft. above high water. Above this level the diameter for the steel shells of the abutment piers for the draw span was reduced to 12 ft. (Fig. 2, Plate XV); for the cylinders under the fixed spans the diameter was reduced to 14 ft. These reductions in diameter permitted of an easy adjustment due to errors in alignment in placing the lower shells (Fig. 5).

To insure a good bond in the cylinder piers at the sections where the 18-ft. diameter was reduced to 12 or 14 ft., fifteen 10-ft. lengths of old steel rail were placed vertically in each cylinder, and spaced around a circumference about 15 in. inside of the upper cylinder. These rails were lashed to the pier girders by steel cables, holes being cut through the webs of the girders where required.

Each of the two rest piers of the draw span consisted of four cylinders (Fig. 1, Plate XVI). Above the water line the 12-ft. sections were braced and tied together by four plate girders, each 96 in. in depth. The two girders running parallel to the track line extended through and were riveted to the farther sides of the cylinders. The

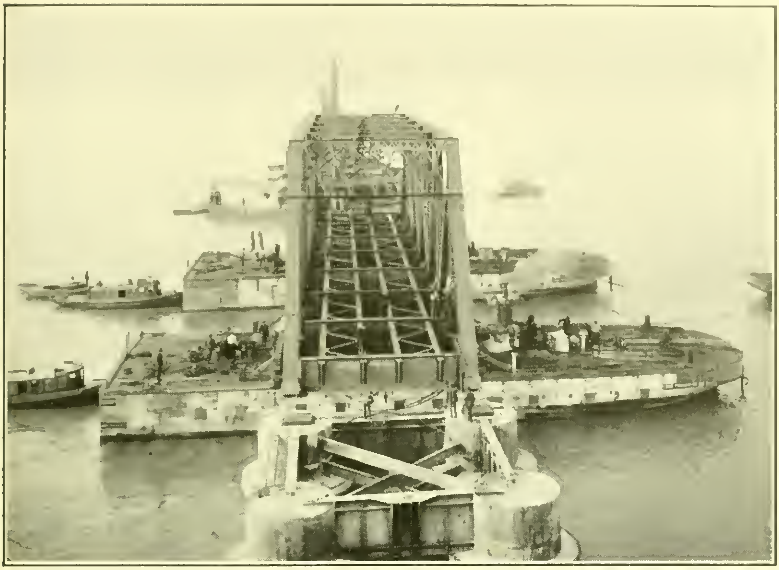


Fig. 1.—Floating Span 5 Into Position, Dumbarton Point Bridge.

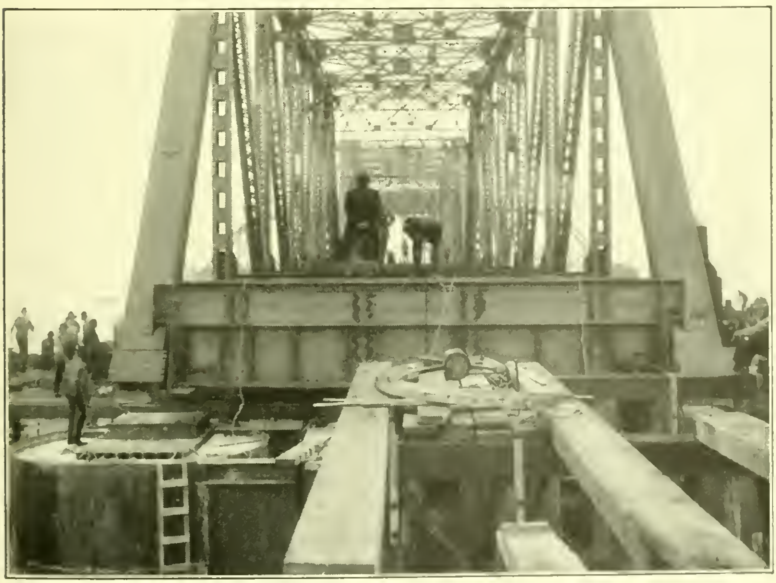


Fig. 2.—Fixed End of Span 1, Dumbarton Point Bridge.

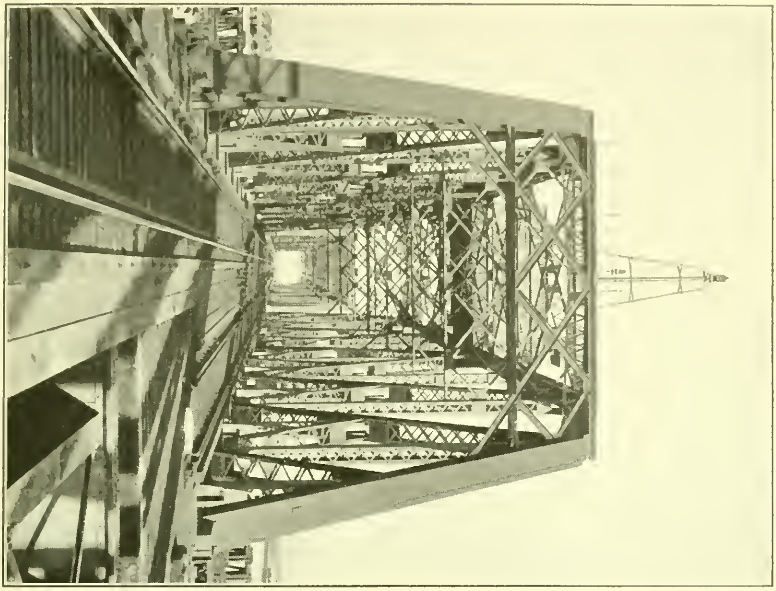


FIG. 1.—DUMBARTON POINT DRAWSPAN.

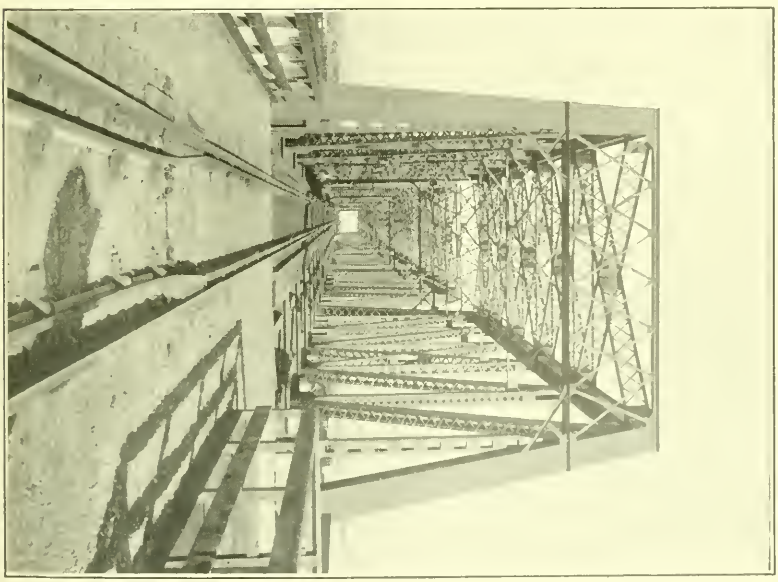


FIG. 2.—DUMBARTON POINT BRIDGE.

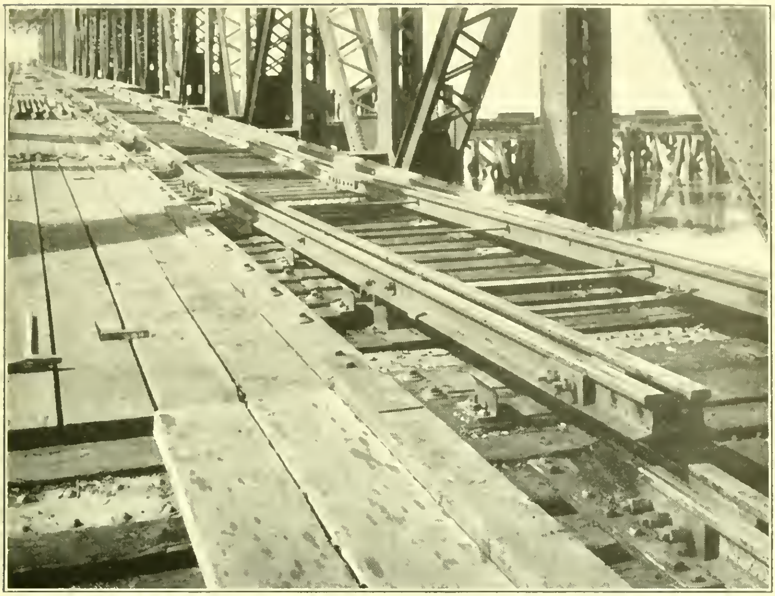


Fig. 1.—Lift Rails, Dumbarton Point Bridge.

Fig. 2.—Dumbarton Point Bridge.

two cross-girders were framed into the longitudinal ones. To provide clearance for placing the grillage beams under the bridge shoes, it was necessary to cut from each end of each girder a portion of the top flange and web.

Each pier under the approach spans consisted of two cylinders, the lower sections 18 ft. in diameter, the upper ones 14 ft. in diameter, as already explained. The 14-ft. sections were tied together by two plate girders, each 7 ft. 2 in. in depth.

The anchor-bolts for all spans were placed as the concrete was poured. The only provision made for adjustment was that provided by the slotted holes of the shoes.

The erection of the draw span required no unusual methods. The material as it arrived from the plant was unloaded along the tracks, so that it could be placed on a barge as required. The unloading derrick (Fig. 2, Plate XVI) was somewhat out of the ordinary in that the A-frame was pin-connected, which permitted it to be raised or lowered, an important and advantageous feature in railroad work. The draw-span trusses, floor, and bracing, were assembled and erected on the draw-span protection in a position at right angles to the line of track. A double derrick or traveler (Fig. 1, Plate XVII), running on 4 lines of rails, enabled the erecting crew to make rapid progress. The span was assembled complete (Fig. 2, Plate XVII), ready for riveting in 14 days. Figs. 1 and 2, Plate XVIII, show the draw span.

The six approach spans were floated into position. To facilitate their erection on land, pile bents were driven in the shallow water of the bay shore on the Newark side. A sufficient number of pile bents of this character were built to permit the assembling of two spans at once (Fig. 1, Plate XIX). When one span was thus assembled and completely riveted, the temporary trusses of the falsework were knocked out and barges were placed in position. The old steamer *Thoroughfare,* which for many years had been used to ferry freight cars across San Francisco Bay, incidentally having been condemned by the Government, was cut in two, bulkheaded, and, as though by the irony of fate, assisted in the erection of the bridge which is destined to relegate to the past the scheme of bay transportation, for which the old steamer was originally built.

Particular care was used in selecting the time to float the various approach spans into position on the cylinder piers. At the erection

site the barges were placed under the spans at low water, so that the rising tide would lift the steelwork clear from the temporary pile supports. The spans were towed against the tide (Fig. 2, Plate XIX). When a span got into position between its cylinder piers (Fig. 1, Plate XX) block and tackle lines were thrown out and made fast to the piers. These lines were connected with donkey engines and capstans, so that a span could be adjusted to its proper position over the grillage beams (Fig. 2, Plate XX) and hold it while being lowered. The lowering was accomplished by opening sea valves in the barge. When the span was in place on the grillage, the cribbing was knocked out and the barge withdrawn. The average time required to place a span, from the time it left the falsework on the shore until the removal of the barge from under it at the piers, was about 2 hours, but in each case depended somewhat on the distance the span had to be towed.

To provide for a continuous rail (Fig. 1, Plate XXII) at the ends of the draw span, the inside flanges of the guide and traffic rails were cut off flush with the web to permit the rails to slip over a cast-steel filler block. The rails were hinged with a slotted hole at the other end where a similar casting was placed. The engine which provides turning power for the draw also operates the end track-lifting devices. The bridge is opened or closed in 2 min. Fig. 2, Plate XXII, is a view of the completed bridge.

About 15 000 cu. yd. of concrete were placed in the foundations; 3 137 cu. yd. were required for the pivot pier. The total weight of steel is 5 935 tons, divided about as follows: draw span, including machinery, 1 215 tons; center pier, 250 tons; six 180-ft. approach spans, 480 tons each; in all approach and rest piers, 1 590 tons.

The work was designed and constructed under the supervision of William Hood, M. Am. Soc. C. E., Chief Engineer, Southern Pacific Company, to whom the writer is indebted for much of the information contained in this paper. The field work was in charge of Mr. W. E. Marsh, Assistant Engineer, until his death, in September, 1909. Too much credit cannot be given to Mr. Marsh for devising with success the many field details and operations which were developed as the work progressed. Mr. C. R. Broughton, his assistant, then took charge of the field work, and to him and to Mr. C. M. Kurtz, who took the photographs, the writer is under obligations for assistance in collecting the facts described in this paper.

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PAPERS AND DISCUSSIONS

A WESTERN TYPE OF MOVABLE WEIR DAM.
Discussion.*

By W. C. Hammatt, M. Am. Soc. C. E.†

W. C. Hammatt, M. Am. Soc. C. E. (by letter).—The writer is glad if his paper has added anything to the knowledge of small inexpensive structures for canals, etc., but does not wish to be understood as recommending the use of timber as a construction material, except in the extremely rare occasions of economic utility. Under ordinary conditions, the cost of reinforced concrete is only 50% more than the equivalent timber construction, and this should generally make its use preferable. Mr. Hammatt.

Mr. Atwood states that this type of dam is more adapted to use in irrigation canals than in streams, due to the likelihood of driftwood in the latter lodging against the bents in time of flood. As a matter of fact, dams of this type are used extensively in rivers, the drift being taken care of by floating log booms arranged so as to conduct it into still water. On the other hand, structures of any type having removable parts which are seated in grooves or sockets in the floor are found to be impracticable in Western rivers because of the deposition of sand on the floor on the recession of the flood. In the case of the flash-board weir, this sand is scoured by the increased velocity on the approach of the board as it is dropped in the grooves, and the board seats itself in a few minutes, regardless of the quantity of sand which has been deposited.

In the rivers of the West the flow conditions differ from those of the East. For example, the flow in the San Joaquin River ranges from about 200 sec-ft. minimum to more than 50 000 sec-ft. maximum. The sudden termination of one of these floods will cause the deposit of sometimes as much as 3 ft. of sand on the floor of the weir.

* Continued from November, 1912, *Proceedings*.

† Author's closure.

Mr. Hammatt.

Regarding the question of scour: In general, the weir floor is made long enough so that there is not much tendency in that direction. If the length of the floor is such that a line from the top of the weir, making an angle of 45° with the horizontal, falls within the limits of the floor, there is little tendency to scour. Generally, there is no occasion to put in the small auxiliary bulkhead shown in the writer's plan, as a sand-bar almost always forms a short distance below the weir, which has the same effect, as it backs up the water and forms a water-cushion to break the impact of the overpour. The same effect is obtained by placing the elevation of the weir floor 1 or 2 ft. below the grade of the canal or river bottom.

The expedient suggested by Mr. Atwood, namely, that of sloping the floor upward, would have an effect the opposite of that desired. The velocity of a certain volume of water falling over a vertical weir is less than that of the same volume falling over a weir inclined down stream. Consequently, the scour would be less in the former case.

In conclusion, it may be stated that this type of structure, built of the material which will give the maximum service, and with the substructure best adapted to the foundation on which the weir is to stand, is probably the least expensive of any of the movable types for waterways where the floods are flashy and a large, and at the same time accurate, regulation is desired.

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PAPERS AND DISCUSSIONS

THE SIXTH AVENUE SUBWAY OF THE HUDSON AND MANHATTAN RAILROAD.

Discussion.*

By H. G. Burrowes, M. Am. Soc. C. E.†

Mr. Burrowes.

H. G. Burrowes, M. Am. Soc. C. E. (by letter).—In regard to the water-proofing, the membranous method had been used between 12th and 27th Streets, and had not given entire satisfaction, as leaks had developed after completion; therefore, in order to obtain better results, Keystone Water-Proofing Compound was used experimentally between 27th and 33d Streets. In general, it was satisfactory, except at stations, because a leak which would be unnoticed between stations, would be very unsightly and objectionable at a station.

As it was necessary to construct the subway in alternate slices or sections, first under the elevated column bents and then between these bents, there were numerous joints, and it was there that the leaks occurred, and also at places where structural steel extended through the concrete or where reinforcing rods ended close to its inner face. These leaks were stopped by cutting away several inches of concrete from the inside and then plastering with cement mixed with water-proofing compound, by grouting, and, in some cases, where the back-fill had not been completed, by stripping the roof and then placing brick laid in hot asphalt.

The cost was much less than with the membranous method, for not only was the cost of the compound less than that of the membrane, but the outer protecting masonry and the additional excavation for it became unnecessary; even by adding the amounts spent on stopping leaks, there was an ultimate saving.

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* Continued from November, 1912, *Proceedings*.

† Author's closure.

Mr. Burrowes.

It is very doubtful whether better results could have been obtained by the use of additional cement, as the leaks were due to the joints in the concrete between sections as built and to the presence of steel which extended through or nearly through the concrete roofs.

Under the practical conditions that prevail in executing such work, it is probable that some leaks would have occurred, no matter what class of water-proofing had been used. The conclusion reached was that the use of the compound was satisfactory between stations, but that at the stations the water-proofing results were not positive enough.

The decision to use reinforced concrete instead of structural steel was made for the reason that there is a slight saving in quantities due to the use of the former, as stated by Mr. Whitney; and, in addition to this, the unit cost of the rods was less, and at the time that this structure was designed, prompt delivery of structural material could not be obtained, while rods could always be had.

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PAPERS AND DISCUSSIONS

STATE AND NATIONAL WATER LAWS, WITH DETAILED STATEMENT OF THE OREGON SYSTEM OF WATER TITLES. Discussion.*

By E. G. Hopson, M. Am. Soc. C. E.

E. G. Hopson, M. Am. Soc. C. E. (by letter).—This paper deals with more advanced topics, in connection with the control of National and State waters, than the average man is qualified to follow closely or to endorse in their entirety. Mr. Lewis does not write on these matters as a novice or a mere theorist, but as a responsible State official who has administered successfully the regulation of State waters for many years. Mr. Hopson.

It is generally admitted, by those qualified to judge, that the water laws of Oregon are among the most carefully devised and effective of any in force in the United States. They contain certain admitted defects, more particularly in connection with the control of water-power, which may be expected to be eliminated during the next few years, as experience ripens; but the general provisions governing diversions, application to beneficial use, and appurtenancy to lands, are generally beyond serious criticism, except in details.

It is premature, of course, to claim too much for the Oregon system of water titles and control, their application being as yet quite young. Their ultimate value and success depend almost wholly on the capacity and integrity of the officials whom the State may appoint. Very few officials have had better experience than Mr. Lewis in water-right matters, or have devoted more energy and care to the numerous complex problems that are continually arising therefrom. He has been both an appointed and popularly elected State officer, having especial jurisdiction over the interests and prerogatives of the State in its control of State waters. All his official associations and connections

* Continued from December, 1912, *Proceedings*.

Mr. Hopson. necessarily have tended to emphasize in his mind the doctrine of State control of water. Therefore it is especially significant when a man of this type comes forward to affirm the importance and necessity of radical augmentation of Federal power in the control of public waters.

The Oregon Water Laws, like similar laws in other States, affirm the ownership by the State of all streams within the State, and establish rules governing their use; but an administrative State officer in charge of the practical application of these rules finds himself faced with apparently insoluble inconsistencies. Taking the State of Oregon as typical, there are the following complications: First, the national forest reservations, mainly occupying the great mountain ranges, in which most of the sources of the rivers lie, are under the exclusive and permanent administration and ownership of the United States. The appropriations and uses of water in these reserves, from every practical standpoint, rest wholly with the United States, and not with the State, notwithstanding the doctrine of State control of all waters within the State. It will be readily understood by engineers that the Federal control of storage and diversions within these reservations necessarily conflicts with and renders partly non-effective any system of State control lower on the streams.

Then again, there are the various reservations of reservoir sites and power sites under the Reclamation Act and for the conservation of water-power. These continually run counter to the obvious intent of the administration of the State laws controlling water use, because a State permit to use water for irrigation or power is useless in a large proportion of cases without the utilization of the sites embraced in the reservations.

Then, again, there are the indefinite rights of the Indian reservations, which in many cases are claimed to be superior to rights granted under the State laws to white settlers. These offer a complication not so serious as some, but certainly embarrassing.

Next there are inter-state streams, which meander to and fro across the borders of the State, and on which adverse appropriations are made, first in one State and then in another, both being under absolutely independent State jurisdictions.

Then there are the navigable waters, under an indefinite control by the National Government. It is evident to any thinking person that control of the navigability of a stream necessarily implies some measure of control of the feeders of that stream. If the Government is to spend millions to deepen channels, build locks, dams, and other improvements, and be responsible for the maintenance of deep water for navigation, it is evident that it cannot permit private parties at their pleasure, even under State permits, to make diversions adverse to navigation interests. Hitherto, this question has not assumed the

pressing importance that it will in years to come, because the population of the Western States and its utilization of water are still comparatively small; already, however, at certain points, there are unmistakable indications of a clash between the opposing interests of navigation and State-regulated appropriations. On one important California river, navigation is being seriously threatened by such diversions, and there is no body, either Federal or State, which has power to make an expert decision on the merits of the case in the public interest. In the meantime, important private interests are being permitted to grow up, with the sanction of the State and under the eyes of the Federal officials having charge of navigation, which inevitably must result in serious complications and losses in the future, but which both State and Federal officials are powerless to prevent. Mr. Hopson.

The greater part of Oregon is within the water-shed of two great navigable rivers, the Columbia and the Willamette. On both these streams the Government is expending large sums of money in navigation improvements. The Willamette is an intra-state river; the Columbia carries drainage from all the Northwestern States, as well as from Canada. The summer navigation of both may be impaired to a marked degree when the various diversions believed to be feasible and desirable have been completed.

Serious differences of opinion are also arising from time to time over the uses to which inland inter-state navigable bodies of water should be put, whether to be used for reservoirs, drained and reclaimed, or continued as mere navigable lakes. At present special Congressional action is necessary in each case, and is an uncertain and unsatisfactory recourse.

It is probable that in the majority of cases public interests will be best served by giving preference to diversions for beneficial use over use for navigation. The point of interest is that each case be considered on its merits at the proper time, and a definite policy be laid down by some competent authority, which obviously cannot be a creation of the State.

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PAPERS AND DISCUSSIONS

TUFA CEMENT, AS MANUFACTURED AND USED ON THE LOS ANGELES AQUEDUCT.

Discussion.*

By Messrs. Clifford Richardson, Luther Wagoner, E. D. Knap, and O. E. Mogensen.

Clifford Richardson, M. Am. Soc. C. E. (by letter).—The title of this paper might well have been puzzolan cement, instead of tufa cement, in order to make it more intelligible to the cement expert. From a geological point of view tufa is, no doubt, a more accurate designation of the material in use in California than puzzolan, but the latter is commonly used, as witnessed by the title of Part VII of Eckel's "Cements, Limes and Plasters," which is "Puzzolan Cements." Eckel includes among puzzolanic materials "all those natural or artificial materials which are capable of forming hydraulic cements on being simply mixed with lime, without the use of heat." It is stated by Mr. Lippincott that by mixing the ground California tufa with lime, without any cement, it would set under water and slowly become hard. The material, therefore, may be recognized as being puzzolanic. From the analyses of the tufa in use in California, it appears that those denominated Fairmount and Monolith contain a much higher percentage of silica than is found in the puzzolans from Italy and France, and in the trass from Germany, the percentage of silica in the Italian materials varying from 39 to a maximum of 63, in those of France from 31 to 48, and, in the case of the trass from Germany, 46 to 67.6, according to Eckel. The California material corresponds more closely in composition to santorin earth, which carries as much as 73%, and averages 66.4%, of silica. Unfortunately, no data are available showing what proportion of the silica in any of the materials occurring abroad, or in those in California, is soluble Mr. Richardson.

* Continued from December, 1912, *Proceedings*.

Mr. Richardson. in acid, as it is hydrated silica of this description which possesses the greatest hydraulic value. There seems to be no question, however, but that the ground mixture of Portland cement and California tufa produces a true puzzolanic cement.

It appears that the tufa in California is ground and then blended with the Portland cement. It is natural to suppose that the tufa cannot be economically ground unless it has been thoroughly dried, in view of the general experience in the cement industry with grinding slags and other raw materials. To the writer it would seem that it would have been better practice to grind the tufa and the Portland cement clinker together at the same time, in the same mill, as a more intimate mixture of the materials would result.

One fails to find in the paper the results of any tests of the tufa cement with the local sands in use, either in tension or compression. Data of that description would be instructive, especially as it has been recognized, at least in Germany, that tensile strength tests of cement are of small importance as compared with those in compression, and, on this account, have been abandoned there.

Mr. Lippincott states that:

"A striking feature of the tufa cement is that, in all the four years in which it has been tested, there has never been a pat which has failed under the boiling test. This indicates, further, that any free lime which may occur in the cement combines with the silicas in the tufa."

To the writer, this does not seem to be a conclusion which can be justified. It merely means that the cement has been ground finer by passing it through the Gates tube mill after the two materials have been blended, and, consequently, that it does not contain the coarse particles which cause pats to fail under the boiling test, a test which the best German authorities consider of very little value, and which seems to the writer to bear no relation to the conditions to which the cement is exposed in actual construction.

It appears that, in its Pittsburgh laboratory, the U. S. Bureau of Standards has determined the amount of insoluble silica in the cements and in the tufa, but, unfortunately, the percentages of soluble silica are not stated in the paper, although there is sufficient evidence that there is a considerable percentage of silica in the tufa in a hydrated condition which, in the cement in a set condition, is in the soluble form, although there is no evidence that it is combined with lime.

It would be interesting to determine in some manner the hardness of the puzzolan cement in concrete and its resistance to attrition and impact, as compared with Portland cement. It is frequently deficient in this respect, and is not always as satisfactory as concrete made with Portland cement. Mr. Lippincott is quite right in stating that

a cement of this type is more satisfactory when used in moist places and in masses where it cannot dry out, than where the latter is possible and, on this account, work done with it requires greater care in order to permit of satisfactory setting than that done with straight cement. Mr. Richardson.

Based on the writer's experience with sand cement, it would seem that there was plenty of justification for using tufa cement in the California work, but it will be interesting to watch its behavior during a long period of years.

LUTHER WAGONER, M. AM. SOC. C. E.—Influenced by the reports of the successful use of tufa by the Los Angeles Aqueduct Board, the speaker located several tufa claims for a contemplated work in Arizona, and until recently supposed that the successful use of tufa was definitely settled. In 1911, some parties desired to build a dam and other works near Prescott, Ariz., which would require large quantities of cement. The speaker furnished them with samples of his tufa and was informed that they had also used these and other near-by samples in combination with Portland cement from Iola, Kans., and Portland, Colo., and that all the samples thus tested gave poor or negative results. Mr. Wagoner.

On the speaker's return homeward, he compared samples of the Arizona tufa with those of California tufa, in the Aqueduct office at Los Angeles, the samples appearing to be nearly the same in physical characteristics. Both seemed to be rhyolite tufa.

In June, 1912, he received a copy of a "Report on Municipally Manufactured Cements, Los Angeles Aqueduct," signed "Gervaise Purcell, W. H. Sanders, F. C. Finkle, Chester B. Loomis, Consulting Engineers." This was a report made to Mr. Edward M. Hagar, President of the Association of American Portland Cement Manufacturers, Philadelphia, Pa., in which they attacked vigorously the results obtained by mixing tufa with the cement made by the Aqueduct engineers. They claim that the hydraulic index of the municipally manufactured cement is as low as 1.66 to 1.69, while average American Portland cement has an index averaging 1.91, and German brands 1.98; and they also claim that the tufa used is the physical equivalent of clay, hence the poor results found by them.

Thus far, the speaker has not heard of any public denial by the Aqueduct Board of the truth of the above quoted charges, and the whole matter is thus enveloped in doubt. It would certainly be a matter of considerable interest to the Profession to have careful tests made of the said California tufa, using a standard brand of cement, so that this element of uncertainty as to hydraulic index might be removed, and at the same time settle the question, whether the real benefit is not due so much to the character of the adulterant as to the fact that the regrinding with the tufa brings a larger percentage of the otherwise inert, non-setting grains of Portland cement into a finer

Mr. Wagoner. state, whereby, when wetted, they "gel" or enter into a colloidal condition.

Concerning the incidental matter of fine grinding, the speaker at one time made tests on a certain fine-grained ore carrying silver chloride, the pan tailings assayed 12 oz. per ton, and 11% of the silver was soluble in "hypo"; 1 gramme of the tailings was ground for 1 hour in an agate mortar with "hypo", and every effort was used to crack and break up the grains of mineral, the result being that 50% of the silver was made soluble. On washing and separating the coarsest grains, the latter were found to have a maximum diameter of $\frac{1}{100}$ mm. This test shows that the finer grains probably have a cushioning effect which protects the larger ones from effective action, and this raises the question: Could not the present methods of grinding clinker be greatly improved by using a gradual reduction method in which the fine dust would be removed more promptly, and thus enable much finer grinding of the cement, and probably with greatly improved efficiency, by enabling a larger percentage to act as colloids?

With his present information on the matter, the speaker considers that there are still several points to be cleared up before venturing on the extensive use of tufa as an adulterant or adjunct to Portland cement.

Mr. Knap. E. D. Knap, Assoc. M. Am. Soc. C. E.—The speaker has looked up some records relative to one phase of this tufa cement question, that is, of Portland cement ground with varying percentages of silica, which makes a compound not unlike that of tufa, in that the main ingredient of the latter seems to be a form of silica. These records show the results of tests up to 6 months for mixtures in which the Portland cement ranges from 100% down to $12\frac{1}{2}$ per cent. The strength of the 100% mixture starts with 417 lb. at 7 days, and runs up to 650 lb. A Portland cement ground with an equal weight of white quartz, making a 50% cement, starts at 270 lb., at 7 days, and runs up to 653 lb. at 6 months, which is 201% of the strength of the neat cement, considering the cement content of the mixture.

There is another compound which consists of the second, that is, the 50% Portland cement, and 50% sand, one part each of sand and cement ground together, and another part of crushed quartz mixed with it mechanically. This compound started at 257 lb. and ran up to 614 lb. in 6 months, which is 283%, counting the Portland cement at $33\frac{1}{3}$ per cent.

There was still another mixture which was one part each of cement and sand ground together, with two parts of quartz added, making a 25% mixture. This mixture started at 264 lb. and went to 585 lb., which is 360%, considering the quantity of cement it contained.

From these figures, it would seem that the chief thing to be gained by adding and grinding together silica of any kind and Portland cement,

is first to provide a filler, and secondly to increase the impalpable portion of the Portland cement, because everybody knows that only a small part of the cement in the average Portland cement is of any use, and that is the impalpable powder. The remainder acts as a filling to hold it together, and by grinding quartz and cement together, the binding proportion is very much increased by the additional fine grinding, which cannot be obtained in any other way, at least, it cannot be obtained commercially in any other way. Mr. Knap.

For instance, in addition to this 25% mixture, there is another 25% mixture of the same brand of Portland cement mixed with three parts of standard crushed quartz sand, as in ordinary mortar, and that starts at 134 lb., as against 264 lb., and only runs to 388 lb. at 6 months, which is very much less than 585 lb.; and it seems to the speaker that this "diluted" cement, this tufa, is beneficial chiefly in that for a certain quantity of cement used, a very much greater strength is obtained because of this grinding, as the fine content fills the voids. For instance, the strength of the 25% mixture of sand and cement in the ordinary way as mortar, and the strength of the corresponding mixture partly ground together, is as 1 to 1½; that is, the mixture ground together is 151% of the strength of the ordinary mortar; therefore, it would seem that, aside from the possible combination of some of the free lime, or the extra lime, with the soluble silica of the tufa, as against the insoluble silica of the quartz, probably the greatest advantage is derived from the extra grinding.

The speaker has also the results obtained with a 12½% mixture, a mortar made with three parts of standard crushed quartz sand and one part of the 50% "ground together" mixture previously mentioned, which, at 6 months, shows a strength of 126%, considering the cement content. This mixture, however, starts at only 96 lb. at 7 days and takes 28 days to reach the minimum strength allowed for 7 days. Obviously, this would unfit it for use in any work requiring early strength, no matter how strong it might become later; and this same trouble, namely, the lengthening of the time required for work to "stand alone," seems to be one of the strongest arguments against "diluted" cements in general. They all seem to be good enough in the course of time, but time is often a governing factor. This is true, not only in the case of form work, where every day that the forms have to be left in place causes additional expense, but even more so in cases where frost or bad weather must be considered.

The time consideration applies somewhat to the remarks of others in reference to fine *versus* coarse sand. A sand which will give great ultimate strength may be unfit for use in many cases, because immediate strength also is required. The speaker's experience with different sands, however, leads to the conclusion that, in all cases, it is not accurate to say that a fine sand is poor and a coarse sand is good. He

Mr. Knap. has found good fine sands and poor coarse ones. The measure of efficiency in any sand, it would seem, is in the difference in the relative sizes of the grains composing it, rather than in the actual maximum size of some of the grains; or, expressed differently, in the absence of voids, which are likely to be much less in a sand having grains with a great range of sizes than in one in which the sizes are more uniform.

Mr. Mogensen. O. E. MOGENSEN, M. AM. SOC. C. E.—During the past two years more or less comprehensive articles have appeared in the technical periodicals mentioning this half-and-half mixture of cement and tufa used on an extensive scale on the Pacific Coast. The subject is an important one, of interest to engineers throughout the United States, and the speaker, therefore, was pleased to see it described so thoroughly and comprehensively before this Society.

The high results in tensile strength and the generally good behavior of the laboratory tests, described so fully in this interesting paper, are obtained partly by the chemical reaction and partly by the process of re-grinding in the tube-mills the straight Portland cement mixed with tufa to a fineness greater than that generally required for Portland cement. By this process the two materials are mechanically and intimately mixed and pulverized, which, of course, could not be done on the mixing board. This applies to tufa, or puzzolan, or to slag.

The question as to why fine sand, which is generally known to produce an inferior mortar as compared with coarse sand, in this instance gives such high tensile strength, is readily explained, and is really the secret of the whole process; it is due to the re-grinding and intimate mixing process in the grinding mills.

The combination of Portland cement with a certain percentage of puzzolan is a subject which occupies many minds at the present day; and, whether it be as tufa cement, slag cement, sand cement, or cement mixed with diatomaceous earth, the speaker agrees with the author in his statement: That the consumer should derive the benefit, if a product can be furnished which is cheaper in cost and as good in quality. To this the speaker would add: and provides the same degree of safety to structures occupied by people, or to structures otherwise dangerous if insecure. It must be borne in mind that a large percentage of the cement manufactured, perhaps the largest, is used for concrete in bulk, and for work of lesser importance, and for this a puzzolan cement will meet all requirements, and in certain cases be more satisfactory than straight Portland.

It is natural that the cement manufacturers on the Pacific Coast should oppose such a radical step as this, but it is certainly a very short-sighted policy when the finished product, the concrete, as claimed

in the paper, has been proved to be free from defects, and continues to give good results. Mr. Mogensen.

Table 1 is a summary of the different classifications of the work. It is not clear from this table that the tufa cement was used in the construction of the 91-ft. dam of the Haiwee Reservoir. The paper describes its application for the lining of canals, the covering of conduits, and for pipes, tunnels, and reservoirs. The speaker would like to know if the engineers of the Los Angeles Aqueduct made any discrimination against the use of the tufa cement by excluding it from structures in which possible defects would have serious consequences, causing loss of life. Such discrimination would seem very natural, as no one can predict, even from the most elaborate and carefully conducted laboratory tests, how a new material, such as concrete made from this tufa cement, will stand in the course of years.

Mr. Wagoner's personal opinion and expressions of the views held by some of the engineers on the Pacific Coast are not very encouraging, although not convincing, as they reflect a doubt as to the stability of the work. It is to be hoped that several disinterested engineers who have given this work time and study will contribute liberally to the discussion of this paper. If the author's conclusions, which apparently agree with the results obtained in Europe with like materials, the so-called trass cement, are borne out by the test of time, the knowledge of this material and process cannot be spread too soon for the benefit of the Engineering Profession.

There are, of course, sections of the country and many cement plants where proper materials of this and of similar composition are not available or cannot be obtained at a reasonable price, where no benefit would be derived by deviating from the standard Portland cement practice. This fact, however, should not mitigate against those plants and those communities which would be in a position to benefit by the combination referred to.

Under the heading, "Other Combinations", the author mentions that diatomaceous earth resembles the tufas in their analyses and in physical characteristics, and he gives the result of a set of tensile tests of one-half monolith cement mixed with one-half diatomaceous earth. At the Congress of the International Association for Testing Materials, New York, 1912, a paper on this subject entitled "Diatom-Earth as Puzzolan for Cement," was presented by Mr. A. Poulsen, Chief Engineer, to the Danish Government Maritime Works. In this paper Mr. Poulsen describes the favorable results he has obtained by mixing diatomaceous earth, in proportions determined by chemical analyses, with Portland cement clinker, the two substances being pulverized together in the mill. The cement thus made has shown remarkably good results by producing a concrete which has

Mr. Mogensen. the property of resisting very effectively the action of sea water. Mr. Poulsen's theory agrees with the author's, in so far as the laboratory tests he has conducted show that a chemical reaction takes place between the cement and the diatomaceous earth. The free lime, which he estimates constitutes 30% of the weight of Portland cement, and which in sea water becomes the direct source of disintegrating the concrete by being transformed into calcium sulphate, combines with the silica hydrate and the alumina hydrate in the diatomaceous earth. The diatomaceous earth which he used contains:

Soluble silicic acid (SiO_2)...............	51.6 per cent.
Soluble alumina (Al_2O_3)................	8.5 " "
Making a total of..............	60.1 per cent.

of active substance available for the absorption of the free lime in the concrete.

Mr. Poulsen's paper also contains the results of tests of the tensile strength, of the chemical relations, and the compressive strength of mortars with and without puzzolan. By these tests he wishes to point out the distinct advantage of using diatomaceous earth, as compared with the trass or tufa, and the results agree essentially with those of the author in respect to the advantage of the leaner sand mixture as compared with the richer mixtures.

Mr. Poulsen's statement that the cement when mixed with water contains free lime to the extent of 30% of the weight of Portland cement surprised the speaker, but, in discussing this matter with his associate, Mr. Pontoppidan, Chemical Engineer, the latter confirms the presence of a considerable amount of free lime in mortars made from Portland cement, and explains the chemical reaction which takes place in the following manner:

"This 'free' lime, which is Ca $(OH)_2$, calcium-hydrate, or what is ordinarily called slaked lime, crystallizes as such in long needles, and is important in giving the cohesive strength to the mortar. When Portland cement is mixed with tufa, trass, or 'Moler' (diatomaceous earth), and then made into a mortar with water, the alkali-soluble silica in the tufa, trass, or 'Moler' will enter into combination with the formed calcium hydrate, so that this, instead of crystallizing as such, crystallizes as mono-calcium silicate, after this formula:

$$Ca\,(OH)_2 + SiO_2 + H_2O = CaO,\,SiO_2,\,2\,H_2O.$$

This is the same chemical process which gives the strength to the sand-lime brick, made of 98 parts of sand (silica) and two parts of slaked lime; only, in this case, the silica in the sand is not soluble, and a prolonged storage in superheated steam of 8 atmospheric pressure is required to effect the combination.

"The alkaline water of the desert countries contains, very often, potassium sulphate ($K_2\,SO_4$) and sodium sulphate ($Na_2\,SO_4$); these

salts are undoubtedly destructive to Portland cement in the same manner as is the magnesium sulphate in the sea water, and it is therefore reasonable to assume that the admixture of trass, tufa, or 'Moler' to the straight Portland cement will be as beneficial for concrete work in these countries as it is for sea-water works." Mr. Mogensen.

The paper entitled "The Effects of Alkali on Cement," by George G. Anderson, M. Am. Soc. C. E.,* mentions this very condition, and in the discussion, attention is directed to remedies which may be found by using puzzolan cement rather than straight cement. There is, therefore, good reason not to condemn offhand this adulteration, which for certain purposes may prove superior to straight Portland cement and for others equally well adapted. The speaker is fully aware of the danger and the difficulty opened up by the introduction of two or more standards for cement. These difficulties, however, will gradually be overcome if the combination can prove its right to existence, that is, its technical and economical advantages.

* *Transactions*, Am. Soc. C. E., Vol. LXVII, p. 572.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ECONOMIC ASPECT OF SEEPAGE AND OTHER LOSSES IN IRRIGATION SYSTEMS.

Discussion.*

By Luther Wagoner, M. Am. Soc. C. E.

Luther Wagoner, M. Am. Soc. C. E.—Under the speaker's direction, a concrete lining was applied to a ditch near the Mokelumne River, in California, in 1912. To test seepage loss, a portion of the unlined ditch was filled 2 ft. deep with water; it settled at the rate of 1 in. per hour, and one day later there was a depth of water of 4 in. in the ditch. This loss, in comparison with the total quantity of water carried by the ditch, was small. The chief object of the lining was to avoid the numerous breaks caused by gophers. Mr. Wagoner.

As there was no local supply of concrete material, the best bid obtainable was 9.7 cents per sq. ft. for a 1 : 10 gravel concrete, 2 in. thick. This was applied to the dressed banks and kept wet by watering with a hose. Later, a 1 : 2 grouting was washed over the surface. After two days, a section of the lined ditch was filled 2 ft. deep with water for use on the following section. One week later there was 1 ft. of water in the lined ditch, the loss of 1 ft. being seepage and use. It was not especially desired to cut down the seepage loss, but rather to stop perforation by gophers, and, for this reason, as well as to reduce costs, a 1 : 10 mixture was considered of ample strength to stand up on a 1 to 1 slope, and that it would be gopher-proof.

It was brought out at the Hetch Hetchy public hearing, before Secretary Fisher, at Washington, D. C., on November 25th-30th, 1912, that all unlined ditches sustain a considerable loss. F. H. Newell, M. Am. Soc. C. E., Director of the Reclamation Service, conceded that losses on the Turlock and Modesto, California, ditches and laterals

* Continued from December, 1912, *Proceedings*.

Mr. Wagoner. might reach from 35 to 45% of the water turned into them, and that a 2-in. sand-cement lining might cut down the losses to 15 per cent.

The possible saving of from 20 to 30% of the water by lining would only be an economic possibility when water is high priced, or when it can be used for a high-priced crop like citrus fruits. For the ordinary crops, and present conditions in Central California, it does not appear as yet that there is any economic warrant for lining. As the wealth of the community increases, doubtless this will be done, largely for the purpose of extending the use of the water over a greater acreage, as the areas of good lands are greater than can at present be supplied with water.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

PREVENTION OF MOSQUITO BREEDING.
Discussion.*

By Messrs. Harold Farnsworth Gray, J. J. Rosenthal, and E. P. Felt.

Harold Farnsworth Gray, Jun. Am. Soc. C. E. (by letter).—Within the short space of this paper, the author has compressed considerable information on the subject of mosquito control. It is hoped that the paper will call to the attention of engineers the fact that, in many instances of design and construction, there are details which, if overlooked, may be the cause of considerable annoyance on account of mosquito breeding. Mr. Gray.

The writer gains the impression, on reading this paper, that Mr. Miller's experience with the mosquito problem has been obtained largely in New Jersey and contiguous territory, where it is more a problem of nuisance than of health. Considered on a national scale, however, mosquito control is primarily a problem of health, and is rural rather than urban. In general, it becomes an urban problem only when there is a lax administration of the municipal departments of health and public works.

The author's observations on cesspools as breeding places of mosquitoes have been confirmed by the writer in many cases. A particular illustration of this point came to his attention two summers ago, in a town of about 5 000 population. At that time a sewer system was being constructed, and completion was expected in September. On that assumption, the residents did not keep their cesspools in proper repair, in view of the expected expense of sewer connections in the near future, and a large number of cesspools were in very bad con-

* This discussion (of the paper by Spencer Miller, M. Am. Soc. C. E., published in November, 1912, *Proceedings*, and presented at the meeting of December 18th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Gray. dition. In June a pest of mosquitoes resulted, and it required nearly a month of continuous work to get the situation in hand. Under ordinary conditions, however, cesspools, properly constructed and cared for, do not breed mosquitoes. It is also significant that the Anopheline (malaria-transmitting) mosquitoes do not breed in cesspools, except in rare instances.

Mr. Miller has failed to mention street gutters as sources of mosquitoes. This is a common condition in California, on streets with practically no grade, especially if the gutters are fouled with rubbish. The regular use of a broom would abate the nuisance.

Railroad construction very often diverts or obstructs the natural drainage of the region through which the line passes. The Southern Pacific Railroad through Placer County, in California, furnishes numerous examples. Between Auburn and Roseville on this line the region is intensely malarial, and the railroad contributes a large number of breeding places. The difficulty has been caused by the lack of sufficient culverts, or by culverts improperly placed, and could have been obviated at slight cost at the time of construction. A portion of this line has been greatly improved, partly by ditching and partly by filling on the higher side of the embankments. The writer would call to the attention of engineers the fact that it is not the collection of water in considerable volume and depth behind embankments which is responsible for mosquito breeding. This might occur if no culverts at all were placed under a high fill. It is the small, shallow pools which breed the Anophelines, and, to prevent such slight accumulations, water culverts must be placed more carefully than has been the custom.

A matter not mentioned by the author, and probably the most important in engineering interest, is the eradication of mosquitoes and malaria on irrigation projects. In California, this problem is decidedly prevalent, practically no project in the Sacramento-San Joaquin Valley being exempt. The establishment of irrigation has often been marked either by the introduction, or the noticeable increase, of malaria. Irrigation projects are financed and executed as commercial propositions, and anything that affects unfavorably the sale of such lands means a financial loss in two ways, first, the selling value of the land may depreciate, second, the sales may be retarded, tying up capital for a longer period than anticipated. Malaria is a very potent factor in depreciating the value, and retarding the sale, of land. In regions which are potentially malarial, the engineers in charge of irrigation projects should investigate carefully the possibilities of mosquito breeding, and at the time of construction make such provisions as are necessary to prevent the accumulation of stagnant water. The writer hopes that members of this Society who have had to cope with such conditions will add their data to the discussion of this paper.

It may be stated, as a general proposition, that it is the misuse of irrigation water, or defective irrigation systems, and not irrigation in itself, that is responsible for mosquito breeding and malaria on irrigated lands. The writer need not call to the attention of this Society the fact that irrigation unaccompanied by such drainage as is necessary to remove promptly all excess water is bad engineering and agricultural practice, as well as unhealthful. In California, attention to this matter is apparently seldom given. There are a number of cases on record where whole districts have been ruined by irrigation in which there was no provision for drainage, and in certain large areas the ground-water has been raised to an alarming extent. Under such conditions, mosquitoes and malaria become an important factor in retarding commercial and agricultural development. Mr. Gray.

In the writer's opinion, the solution of the mosquito problem on irrigated projects lies in the following measures:

(1) Improvement, not obstruction, of the natural drainage courses;
(2) Adequate artificial drainage;
(3) Prevention of seepage from ditches, with its resulting collection of water in adjoining low areas;
(4) Proper leveling or surfacing of the land, to eliminate small low areas;
(5) Maintenance of canals, ditches, and drains free from obstructions to flow;
(6) Strict regulation of the use of water, to avoid excessive use and waste;
(7) The occasional use of oil or larvicides on isolated collections of stagnant water which are too small or temporary to require drainage.

With the author's selection of the county as a proper unit of mosquito control operations the writer heartily concurs, but with the condition that the field of the county's operations shall not include incorporated cities. The city problem, as far as local breeding is concerned, is best handled by the city health department. The advisability of a special county mosquito extermination commission is questionable. The writer does not deny that in New Jersey, where the mosquito problem is largely due to salt marshes, a special commission may be advantageous; but the average mosquito problem is very directly one of public health, and, therefore, should be a part of the work of the county health office. The writer is convinced that, as a general rule, the appointment of special commissions is an unnecessary multiplication of executive bodies in the county government. It may be objected that the average county health department is inefficient, and that it does not argue well for the success of anti-mosquito measures to trust them to such a

Mr. Gray. department. This may be granted, but, on the other hand, it may be argued that no department of county government, on the average, is remarkable for its intelligence and efficiency, that there is no assurance that a special commission will show greater efficiency than the existing component parts of the county government, and that the increase of executive bodies under such conditions can only result in duplication of functions and still greater inefficiency. Efficiency in government is to be obtained by concentration, not dispersion, of functions and authority.

In California, in spite of a crying need for anti-mosquito work over a very large area, practically nothing has been done by the county governments. The only attempt in this direction, so far as the writer is aware, has been the passage of an ordinance by the Board of Supervisors of Tehama County. This ordinance was constructed by the writer, and was designed as a club in the hands of a certain irrigation company which is trying, with fair success, to eliminate malaria on its project. The county officials have provided no funds for the enforcement of this ordinance, although the economic loss to this county from malaria is certainly not less than $100 000 annually. The expenditure of $5 000 annually by the county for the administration of anti-mosquito work (the cost of drainage to fall on the owner of the land benefited) would make it possible practically to wipe out this economic loss in 5 years, with a net economic saving in this period of about $330 000.

With the author's statement, that "The civil engineer has an opportunity to serve his fellow men and do a vast amount of good work in the prevention of mosquito breeding, if he acquires the knowledge now available on their breeding habits," the writer can agree only within certain limits. He believes that engineers, simply as engineers, are not qualified to undertake in an executive capacity the direction of anti-mosquito measures of magnitude, as such work, for highly successful prosecution, requires marked abilities as an educator and publicist, as well as a unique combination of thorough knowledge as to engineering, parasitology, hygiene, vital statistics, law, and government. The remarkable abilities of Sir Ronald Ross illustrate the point in question. At the same time, engineers can render distinct and valuable service by making certain that the works they design and construct do not become, through mosquito breeding, a menace to the health and comfort of the people.

Mr. Rosenthal. J. J. Rosenthal, Assoc. M. Am. Soc. C. E. (by letter).—The writer is deeply interested in this paper, as he has experienced much unpleasantness from this pest on Corregidor Island, Philippines, a military post about 3 miles square. The houses are screened, but only by using

mosquito nets on the beds can one insure any sort of protection from these insects at night. Mr. Rosenthal.

About four years ago, before active construction was commenced, and when there were fewer inhabitants, there was no trouble from mosquitoes. The rains, producing pools in natural depressions, were as heavy then as now, so that the influx of the pest must have been due to the numerous uncovered wooden water tanks and barrels introduced by the U. S. Engineers and the Quartermaster's Department, wherever work was going on. An effort is now being made to eliminate the insects by destroying all unnecessary water receptacles and building reinforced concrete covered reservoirs with screens over all openings.

The Island of Caballo, about half the area of Corregidor, and only ½ mile south of it, offers a pleasing contrast. At this place, when construction was begun by the U. S. Engineers, all water receptacles were kept covered, and care was taken to prevent the accumulation of any stagnant water, hence the mosquito has not yet found a home there.

In Manila the pest still breeds heavily, due, in part, to the uncleanliness of the natives and to inadequate sewerage and water connections, especially in the localities where the nipa (palm-leaf) and bamboo huts still abound.

E. P. Felt, Esq.* (by letter).—The studies and observations on mosquitoes in various localities in the State by the writer's department, convince him that the engineer may play a very important part in the control of mosquito breeding, because so much of it is purely local. He is thoroughly convinced, for instance, that the incidental pool, roadside or other, is a more important factor in the production of mosquitoes than the larger bodies of water so frequently viewed with distrust by the general public. The engineer has much to say as to methods, in all undertakings where his services are necessary, and it frequently happens that very little or no additional expense is required to eliminate small collections of water, sometimes ignored by members of that Profession. Mr. Felt.

The writer is particularly interested in Mr. Miller's reference to sewer catch-basins and the possibility of eliminating mosquito breeding in such places. It has occurred to him that, if no better method could be devised, it would be comparatively easy to suspend over the collection of water in the catch-basin, a small container filled with petroleum or other oil and adjusted in such a way as to drip slowly for an extended period. This device, in a modified form, has been used most successfully in certain tropical regions where thorough drainage was impractical. The same idea could be applied to cesspools, and also

* State Entomologist, Albany, N. Y.

Mr. Felt. to cisterns, if the water from the latter was taken from below the surface.

The writer heartily endorses the idea of extending mosquito control over considerable areas. There is more or less migration or drifting of these annoying forms, and, the more extended the work, the better will be the results as a whole. The communities along shore lines, particularly in regions of extensive marshes, are more interested than those farther inland, which may be rarely annoyed by migratory forms, such as the banded salt-marsh mosquito. While it would be desirable to have the unit for mosquito control coincident with county lines, under conditions such as mentioned above, it would seem probable that greater efficacy would be secured by the co-operation of townships or communities suffering from similar conditions. It is difficult, for example, for inland residents to appreciate the desirability of large expenditures for the control of marsh forms practically unknown in their own locality.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE SANITATION OF CONSTRUCTION CAMPS.
Discussion.*

By Messrs. R. C. Hardman and Spencer Miller.

R. C. Hardman, Assoc. M. Am. Soc. C. E. (by letter).—This paper is one of very timely interest, its only fault being its brevity. The writer has had occasion to witness the good effects of camp sanitation, as well as to install certain sanitary appliances in a camp of 4 100 men, under unfavorable climatic and drainage conditions. The subject is one which deserves to be expanded, and should receive the careful attention of the Profession. Mr. Hardman.

In some camps which the writer has seen, the conditions approach those which must have existed in the Middle Ages, and now exist in the less enlightened portions of the world. In the provincial districts of the Philippines, for instance, Americans are horrified to see practically all human waste consumed by hogs kept in the back yards of nearly all the native shacks; yet they view with apparent equanimity the promiscuous deposition of fecal matter around a camp wherein the services of our porcine friends are not available.

The writer once met an American in Hong Kong who solemnly swore total abstinence from fish because he had seen in Canton, China, a fish pond wherein the fish were fed, in part, from a privy built over the edge of the pond. Yet we, at home, empty our raw, untreated sewage into our rivers from which fish are taken for the table, and in thousands of instances privies are built over streams and ponds, as in the case of the Buddhist monastery in Canton; and yet "the pot called the kettle black." The Cantonese did not use the pond for a water supply; we do use our rivers for that purpose.

* This discussion (of the paper by Harold Farnsworth Gray, Jun. Am. Soc. C. E., published in November, 1912, *Proceedings*, and presented at the meeting of December 18th, 1912), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Hardman. To carry the comparison still farther: The writer has seen in Chinese cities huge tubs along the streets in use as "public comfort stations." In a great many cases, these "stations" overflow into the streets; yet this is not much worse than conditions in some construction camps where latrine trenches are dug, left open to the light and air, and flies, and allowed to fill up without destructive or disinfective treatment of any kind.

We think of the Chinese race and nation as a very filthy one, but the filth is only a matter of comparison. The Chinese have yet to be educated up to a high sanitary standard, while we have to be educated to the pre-eminence of health over dollars.

The writer has brought this subject to the attention of the Profession in a short article* containing rules for the use of the various appliances described and illustrated. They differ slightly in form from Mr. Gray's, but cover the same ground. The method, however, is of minor import.

Perhaps the greatest arguments for camp sanitation are found in the records made in the Canal Zone, in Cuba after the Spanish-American War, in the various maneuver camps of the United States army in recent years, and, notably, in the work of the Japanese Medical Corps in the war with Russia. One has but to compare these with the history of the French Company at Panama and the disgraceful record of the U. S. Medical Corps during the Spanish-American War, at Chickamauga, Jacksonville, and elsewhere.

Mr. Gray dwells only on the baser financial side of "the value of sanitation." Of course, this is the only thing which will appeal to the average camp executive, but should we not, as engineers, also be moved by the more altruistic motive of helping the ignorant foreigner, who makes up the bulk of our camp population, to attain, by example and precept, the standard of living of the Americans with whom he is to become assimilated? Do we not owe it as a debt to the human race?

Mr. Miller. SPENCER MILLER, M. AM. SOC. C. E.—Mr. Gray's paper is interesting and valuable.

A Government contractor, building a lock and dam on one of the Western rivers, recently informed the speaker that an outbreak of malaria in his camp had nearly cost him the profit on his contract. Malaria and malarial fever disorganized his camp so completely that he found it practically impossible to keep a full force of men at work. New men arriving to take the places of sick ones would shortly succumb to the disease; others fled from the camp. It was only after this had continued for a considerable time that the contractor sought relief from a sanitary expert. The outbreak of malaria was attributed

* *Engineering News*, October 17th, 1912.

directly to the presence of the malarial mosquito. The camps were thoroughly fumigated and screened. The sick were properly guarded against the mosquitoes. The immediate vicinity of the camp was cleared up. The rank growth was mown down. Ditches were dug and drainage operations were carried out thoroughly. Every receptacle for holding water was either emptied, destroyed, or oiled regularly. Shortly the trouble disappeared. The camp became a healthy and a profitable one. All of which illustrates that it pays to be sanitary. Mr. Miller.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ANDREW BELL, M. Am. Soc. C. E.*

DIED OCTOBER 22D, 1912.

Andrew Bell was born at Toronto, Ont., Canada, on December 21st, 1835. His father, the late Rev. Andrew Bell, was the eldest son of the Rev. William Bell, the pioneer clergyman of Perth, Ont., and the surrounding country. His mother was the eldest daughter of the late Col. E. W. Thomson, of Toronto, a cousin of Baron Sydenham, who was Governor of Upper Canada at the time of consolidation of the provinces.

Mr. Bell was educated at Queen's College, from which he was graduated in 1853 with the degree of B. A. Immediately after his graduation he was articled to Mr. William Syles as a student of Civil Engineering.

In February, 1854, he was appointed Rodman on the survey of the Brockville and Ottawa Railway (now a part of the Canadian Pacific Railway) from Smith's Falls to Carleton Place, and to Perth. This work was discontinued in 1855, and Mr. Bell was engaged in making surveys, under K. Tully, Esq., for the proposed canal from Toronto to Georgian Bay.

In July, 1856, he was appointed Resident Engineer on the Brockville and Ottawa Railway, which position he retained until 1859, having been engaged in the following work: In 1856 he surveyed and located the line from Brockville to Smith's Falls; in October of the same year, he was sent to Almonte, Ont., and placed in charge of the construction, under the late Alphonse Brooks, from Carleton Place to Pakenham; and from 1857 to 1859, he was in charge of the surveys from Sand Point to Pembroke and of the completion of the construction between Carleton Place and Almonte.

Between 1864 and 1869, Mr. Bell had charge of the construction of the large woolen mills at Almonte and Cornwall, Ont., being engaged, at the same time, in general private practice. In 1869 he made a hydraulic survey between Lake Superior and Lake Winnipeg for the Dominion Government.

In June, 1870, he was appointed Resident Engineer on the enlargement of the Grenville Canal, remaining in this position until June.

* Memoir prepared by the Secretary from information on file at the Society House.

1872, when he became Resident Engineer on the construction of the Carillon Canal Dam and Slide. After the completion of this work, in 1886, Mr. Bell removed to Almonte, Ont., where, until his death, he was engaged in private practice as a Civil Engineer, Land Surveyor, and Architect. While engaged on a survey of the country east of Cochrane, on the Grand Trunk Pacific Railway, he was taken ill and was forced to return to his home in Almonte, where he died on October 22d, 1912.

In 1858, Mr. Bell was married to Miss Marion Rosamond, eldest daughter of the late James Rosamond, of Almonte, who, with five children survives him.

Mr. Bell was elected a Member of the American Society of Civil Engineers on September 5th, 1883. He was also a Member of the Canadian Society of Civil Engineers, and many other technical associations.

WILLIAM BELL WHITE HOWE, M. Am. Soc. C. E.*

Died February 11th, 1912.

William Bell White Howe was the eldest child of William Bell White Howe who was originally from Claremont, N. H., and who, after studying for the ministry, went to South Carolina in his early manhood and became Bishop of the Episcopal Diocese of that State.

William Bell White Howe, Jr., was born in Charleston, S. C., on November 2d, 1851. After graduation at Charleston College he applied himself to the Civil Engineering Profession, and, after some preliminary experience on railroad surveys in Texas, was employed as Assistant Engineer on the harbor improvements at Charleston under the United States Engineer Corps. From 1876 to 1879 he was a member of the firm of Simons and Howe, Civil Engineers and Architects, in Charleston, and was Architect of the William Enston Home, in that city, as well as Chief Engineer of the Charleston Bridge Company.

Mr. Howe then became associated, as Assistant Engineer and afterward as Chief Engineer, with the system of railroads extending from Charleston into Georgia, Florida, and Alabama, popularly known as "The Plant System." He was instrumental in the substitution of iron bridges for wooden ones throughout the system, and also in the planning and construction of terminal stations and docks at Charleston, S. C., and at Savannah and Brunswick, Ga. As this system

* Memoir prepared by H. S. Haines, M. Am. Soc. C. E.

of railways was extended in the several States above mentioned, he was Chief Engineer of construction as well.

His ability as an engineer was made evident in the design and construction of the terminal station at Jacksonville, Fla., and more especially in the great work accomplished at Port Tampa, where a deep-water harbor was established on a shoal nearly a mile from shore in Tampa Bay. This harbor and system of docks afforded the best facilities for ocean traffic on the entire west coast of the Gulf of Mexico, and was made memorable as the base of operations against Cuba during the Spanish-American War. As an auxiliary enterprise to "The Plant System", Mr. Howe converted the street railway system in Jacksonville, Fla., into an electric railway, and built a power-house. He also planned the enlargement of the commercial docks at Key West, Fla., and the ocean pier at Palm Beach. Later, he withdrew from the more active pursuit of his Profession, and resided at Spartanburg, S. C., where he interested himself in architectural work and in water-power projects.

Mr. Howe was of that turn of mind which is particularly adapted to a career of civil engineering. With considerable general culture, he was a close student of every branch of the Profession in which he was engaged. His early training as an architect was of advantage in planning important terminal structures, and, in connection with the works at Port Tampa and the Jacksonville Street Railway, he became well qualified as an electrical engineer. His researches into the properties of cement were recognized in his appointment as a member of the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers in 1897.

Only as occasion called for it was Mr. Howe's innate ability made manifest, for a reserve due to modesty screened his attainments from the observation of those who did not know him well; but, by reason of his conscientious discharge of duty and his unvarying kindness of demeanor, his more intimate acquaintances became warmly attached to him; and no one appreciated his personal qualities more than the writer of this memorial, with whom he was closely associated for many years. In the spring of 1911, his failing health caused him to retire to his summer home at Flat Rock, N. C., where he died from cerebral hemorrhage on February 11th, 1912, in the sixty-second year of his age.

Mr. Howe married Miss Louisa Preston King, granddaughter of the Hon. Mitchell King, of the South Carolina bar, who survives him. He is also survived by an only son, Dr. William B. W. Howe, of Hendersonville, N. C.

Mr. Howe was elected a Member of the American Society of Civil Engineers on March 1st, 1893.

WILLIAM HASELTON PUFFER, M. Am. Soc. C. E.*

Died March 17th, 1912.

William Haselton Puffer, the son of the Reverend John M. and Anna Haselton Puffer, was born on May 3d, 1861, in Essex, N. Y. He attended the village schools until his fifteenth year, when, on the death of his father, the family moved to Chelsea, Vt., where he took up the management of a farm and the support of the family.

Mr. Puffer was not able to resume his schooling until he was 22 years of age. He then attended the Bradford Academy and later the St. Johnsbury Academy where he was prepared for college. He entered Dartmouth College in 1887 and the Thayer School of Civil Engineering in 1890, and was graduated from the latter in 1892. His struggles to obtain an education took him past the usual age for graduation from an engineering school, but the time was not lost. His early experience with adversity strengthened his will, broadened his character, and supplied him with a stock of common sense and practical experience which was of great value in his engineering work.

After his graduation, Mr. Puffer was employed on general railroad work with the Atchison, Topeka and Santa Fé Railroad until late in 1893, when he became Assistant Engineer in charge of construction for the Mexico, Cuernavaca and Pacific Railroad. In the summer of 1895 he served as Engineer for Sooysmith and Company, in charge of the foundations for the Cathedral of St. John the Divine, New York City. He returned to railroad work and, until 1897, was in charge of a party on the relocation of center lines for the Boston and Maine Railroad.

In the summer of 1897 Mr. Puffer again entered the employ of Sooysmith and Company and was engaged on the construction of the foundations for the Empire Building in New York City. In the fall of 1897 he entered the service of Joseph W. Cody and Company, as Engineer on shoring and deep foundations. During 1898 he had charge of concrete work for the U. S. Government at Fort Hamilton, and for the next two years he was Chief Constructor for the New York Zoological Society.

In 1901 he was employed by John W. Ferguson as Superintendent on construction of the plant for the Alsen Portland Cement Company, at Alsen, N. Y. After 1902, he was engaged in many mining enterprises in Mexico. He was Manager of the La Tula Mining Company, at Guanajuato, Mexico, from 1906 to 1912, and one of the Directors of the Company from its incorporation. He was also Manager of the Guanajuato Amalgamated Gold Mines Company, in 1909 and 1910.

* Memoir prepared by Arthur W. French. M. Am. Soc. C. E.

While on his way to Carlsbad, N. Mex., Mr. Puffer was stricken with cerebro meningitis, and died, after an illness of two days, in Pecos, Tex., on March 17th, 1912.

In September, 1892, he was married to Helen R. Davis, of Troy, Vt., who, with three children, survives him.

Throughout his life Mr. Puffer impressed friends, employers, and employees with his sterling uprightness of character, his kindness, his ability and good judgment, and his loyalty to duty.

Mr. Puffer was elected a Member of the American Society of Civil Engineers on May 3d, 1905.

JAMES HUGH WISE, Assoc. M. Am. Soc. C. E.*

Died September 16th, 1912.

James Hugh Wise was born in Denver, Colo., on February 27th, 1880. As his father was engaged in mining, the young man spent the first ten years of his life in various parts of New Mexico; he was then brought to San Francisco, Cal. During the next five years he attended the schools of that city and Alameda, and then entered the California School of Mechanical Arts, in San Francisco, from which he was graduated in June, 1899. In that year he entered the University of California, and received his degree from the College of Mines in June, 1903.

During the following year Mr. Wise taught mathematics at the California School of Mechanical Arts, resigning in June, 1904, to enter the employ of the California Gas and Electric Corporation, now the Pacific Gas and Electric Company. His rise to responsibility was strikingly rapid. He continued with the Corporation until February, 1910, when he resigned to become a partner in F. G. Baum and Company, Consulting Engineers. In July, 1911, Mr. Wise again entered the employ of the Pacific Gas and Electric Company, this time as Assistant General Manager, which position he held at the time of his accidental and tragic death. He was on an inspection of the Los Angeles Aqueduct by automobile. While in the southern end of the San Joaquin Valley, it was found that the gasoline tank was leaking, and a stop for repairs was made at a garage. While assisting to empty the tank, it accidentally fell, and Mr. Wise was saturated with the liquid. The day was exceedingly hot, and the gasoline vapor was ignited by a torch burning in a distant part of the garage. Before the flames could be extinguished, Mr. Wise was burned so severely that he died in Oakland the following day.

* Memoir prepared by J. D. Galloway, M. Am. Soc. C. E., and B. A. Etcheverry and H. C. Vensano, Associate Members, Am. Soc. C. E.

In his relatively short career, he had held a number of distinctly responsible positions. Among his earlier works may be mentioned the reconstruction of the Centerville Plant of the Pacific Gas and Electric Company, which consisted of the enlargement of the conduits, the rebuilding of the power station, and the installation of a turbine operating a 5 000-kw. generator under a 600-ft. head. Later, he had charge of the erection of a 5 000-kw. generator at Colgate, and of similar work for the Deer Creek Plant of the Pacific Gas and Electric Company. In 1909, the Lake Arthur Dam was constructed under his supervision.

At the time of his death, Mr. Wise was in direct charge of the design and construction of a large hydro-electric power plant for his Company. This work included a masonry dam, more than 300 ft. high, to store the waters of Lake Spalding. The conduits, pipe lines, buildings, machinery, and steel tower transmission line of the plant were proportioned to generate 40 000 kw.

Mr. Wise, on account of his kindly nature, was uniformly liked by all who met him. He had exquisite tact, most courteous manners, and, in a very high degree, the power to command loyal service and friendship from others. The writers, speaking for themselves and for many who knew him, remarked with pleasure his rapid rise to a responsible position, and they take this occasion to express their deep respect for the noble character of a friend whose death is a cause for profound sorrow and regret. He was a Mason, was unmarried, and is survived by his mother who lives in Berkeley, Cal.

Mr. Wise was elected an Associate Member of the American Society of Civil Engineers, on February 6th, 1907, and, at the time of his death, was Vice-President of the San Francisco Association of the Society.

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OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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VOL. XXXIX—No. 2

FEBRUARY, 1913

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1913

American Society of Civil Engineers

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER....................5913 Columbus.

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Vol. XXXIX. FEBRUARY, 1913. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

SIXTIETH ANNUAL MEETING*

January 15th, 1913.—The meeting was called to order at 10 A. M.; President John A. Ockerson in the chair; Charles Warren Hunt, Secretary; and present, also, about 400 members.

Messrs. C. A. Tinker, W. F. Strouse, W. M. Kinney, and E. G. Kastenhuber, Jr., were appointed Tellers to canvass the Ballot for Officers for the ensuing year.

The Annual Report of the Board of Direction, and the Annual Reports of the Secretary and of the Treasurer,† for the year ending December 31st, 1912, were presented and accepted.

* A full report of the Sixtieth Annual Meeting is printed on pages 68 to 102 of this number of *Proceedings*.

† For these reports, see pages 10 to 20 of *Proceedings* for January, 1913 (Vol. XXXIX).

The Secretary presented the report* of the Board of Direction awarding the prizes for the year ending July, 1912, in accordance with the report of the Committee to Recommend the Award of Prizes, as follows:

The Norman Medal to Paper No. 1212, "The Detroit River Tunnel," by Wilson Sherman Kinnear, M. Am. Soc. C. E.

The Thomas Fitch Rowland Prize to Paper No. 1210, "Reinforced Concrete Bridge Across the Almendares River, Havana Cuba," by Eugene Klapp and W. J. Douglas, Members, Am. Soc. C. E.

The Collingwood Prize for Juniors to Paper No. 1213, "A Reinforced Concrete Stand-Pipe," by W. W. Clifford, Jun. Am. Soc. C. E.

The following were appointed members of the Nominating Committee to serve two years:

S. W. Hoag, Jr.	*Representing District No. 1*
Charles T. Main	" " " *2*
E. A. Fisher	" " " *3*
Richard Khuen	" " " *4*
J. B. Berry	" " " *5*
Frank M. Kerr	" " " *6*
William Mulholland	" " " *7*

A Progress Report of the Special Committee on Concrete and Reinforced Concrete was presented by Richard L. Humphrey, M. Am. Soc. C. E., Secretary of that Committee.†

The report was accepted and the committee continued.

Desmond FitzGerald, Past-President, Am. Soc. C. E., Chairman of the Special Committee on Engineering Education, presented a Progress Report of that Committee.‡

The report was accepted and the committee continued.

Austin L. Bowman, M. Am. Soc. C. E., Chairman of the Special Committee on Steel Columns and Struts, presented a Progress Report of that Committee.§

The report was accepted and the committee continued.

A. H. Blanchard, M. Am. Soc. C. E., Secretary of the Special Committee on Bituminous Materials for Road Construction, presented a Progress Report of that Committee.||

The report was accepted and the committee continued.

Frederic P. Stearns, Past-President, Am. Soc. C. E., Chairman of the Special Committee on the Valuation of Public Utilities, presented a Progress Report of that Committee.¶

* See page 69.
† See page 117.
‡ See page 169.
§ See page 170.
|| See page 175.
¶ See page 176.

The report was accepted and the committee continued.

The Secretary reported that the Board of Direction had appointed a Special Committee to Investigate the Conditions of Employment of, and Compensation of, Civil Engineers, consisting of Alfred Noble, Chairman; S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, and John A. Bensel.

The Secretary reported that the Board of Direction had decided to appoint a Special Committee of seven to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., but that, as yet, the committee had not been completed.

The Secretary presented a report from the Board of Direction in reference to a proposed amendment to the Constitution, dividing the territory occupied by the membership into thirteen geographical districts, instead of the seven which are now prescribed.*

It was moved, seconded, and carried "That the recommendation of the Board be approved."

It was moved, seconded, and carried "That it is the sense of this Meeting that a change as outlined by the report read by the Secretary is desirable; that the Board, in reporting back such an amendment to the Society, remove the restriction as to State and Territorial lines."

It was moved, seconded, and carried "That the whole matter be referred back to the Board of Direction, with request that they prepare what they consider a suitable amendment in consideration of the discussion we have had here to-day, in time to be acted upon by the next Convention."

The Secretary presented a letter from E. W. Clarke, M. Am. Soc. C. E., offering the following resolution:

"That the President appoint a committee of five to investigate and report at the regular business meeting in March, 1913, on the possibility of securing suitable engineering entertainments to take the place of the reading of the formal papers for the semi-monthly meetings."

It was moved, seconded, and carried "That a committee of five be appointed to consider and report upon an improvement of the methods of the presentation of papers before the Society."

The Secretary read a letter from Percival M. Churchill, Assoc. M. Am. Soc. C. E., offering the following resolution:

"*Moved:* That the Board of Direction be instructed to confer with the officers of the Am. Institute of Consulting Engineers, the Am. Soc. of Mechanical Engineers, the Am. Institute of Electrical Engineers, and of any other Engineering Society it deems proper to include, for the purpose of ascertaining if these Societies will join with us in appointing a joint Committee to draw up a plan for the establishment and operation of an Exchange for the Marketing of Engineering

* See page 77.

Services of every sort. The Board of Direction to be authorized and instructed to appoint the Committee from this Society as soon as any of the other Societies signify their willingness to join in the work."

On motion, duly seconded, Mr. Churchill's resolution was referred to the Board of Direction.

The Secretary read another letter from Mr. Churchill proposing the following resolution:

"*Moved:* That this Society shall consider the adoption of a Code of Ethics for Engineers recently proposed by a Committee of the American Society of Mechanical Engineers and published in *Engineering News,* January 2d, 1913.

"That the proposed Code be printed as a letter ballot and submitted to the members of this Society, with the request that each member vote to accept or reject the Code article by article; that where a member so desires he shall—after voting against a certain article—submit a substitute in the form he desires that article to take. In the same manner additions may be presented.

"This ballot to be closed April 1st, 1913.

"The Board of Direction shall then send out a second ballot giving the Code as first proposed, and also any suggested changes and additions. Members shall vote article by article. The ballots to be opened at the next Annual Convention, and any article having a majority of the votes cast shall be declared adopted.

"The Code as thus adopted to be then printed in the *Proceedings* and the *Transactions* of the Society."

On motion, duly seconded, Mr. Churchill's resolution was referred to the Board of Direction.

The Secretary presented a report giving complete information as to the arrangements made thus far for the proposed International Engineering Congress to be held in San Francisco in 1915.*

H. M. Wilson, M. Am. Soc. C. E., offered the following motion:

"That the Board of Direction be and is hereby authorized to consider plans for increasing the capacity of the Society House to better accommodate the membership at the Annual Meetings."

The motion being duly seconded, was carried.

H. H. Quimby, M. Am. Soc. C. E., offered the following motion in reference to the acoustics of the Auditorium:

"That the Board of Direction be requested to employ the best talent that is available, and do something to make it more satisfactory to the hearers in this room and to the speakers also."

The motion, being duly seconded, was carried.

The Secretary presented the report of the Tellers appointed to canvass the Ballot for Officers for the ensuing year.

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* See page 97.

The President announced the election of the following officers:

President, to serve one year:

GEORGE F. SWAIN, Cambridge, Mass.

Vice-Presidents, to serve two years:

J. WALDO SMITH, New York City.
CHARLES H. RUST, Victoria, B. C., Canada.

Treasurer, to serve one year:

JOHN F. WALLACE, New York City.

Directors, to serve three years:

HENRY W. HODGE, New York City.
JAMES H. EDWARDS, Passaic, N. J.
LEONARD METCALF, Boston, Mass.
HENRY R. LEONARD, Philadelphia, Pa.
EDWARD H. CONNOR, Leavenworth, Kans.
SAMUEL H. HEDGES, Seattle, Wash.

Mr. Stearns and Mr. Noble conducted Mr. Swain, the President-elect, to the chair.

Mr. Swain addressed the meeting briefly.

Adjourned.

SPECIAL MEETINGS FOR TOPICAL DISCUSSION ON ROAD CONSTRUCTION AND MAINTENANCE.

January 17th, 1913.—The first special meeting for topical discussion on "Road Construction and Maintenance" was called to order at 10 A. M.; President George F. Swain in the chair; Charles Warren Hunt, Secretary; and present, also, 140 members and guests.

The Secretary announced that Arthur H. Blanchard, M. Am. Soc. C. E., would act as Secretary.

In the absence of Paul McLoud, M. Am. Soc. C. E., who was to have introduced the discussion on the first topic, "Cement-Concrete Pavements," the opening remarks were made by J. A. Johnston, M. Am. Soc. C. E. The topic was discussed further by Messrs. Sanford E. Thompson, W. A. McIntyre, W. M. Kinney, Charles W. Leavitt, Jr., E. H. Thomes, R. A. MacGregor, James Owen, Philip P. Sharples, L. P. Sibley, C. W. Ross, R. W. Lesley, Charles P. Price, S. Whinery, Harold Parker, W. P. Blair, W. A. Aiken, Percy H. Wilson, and Arthur H. Blanchard.

Adjourned.

January 17th, 1913.—The second special meeting was called to order at 2.30 P. M.; Harold Parker, M. Am. Soc. C. E., in the chair; Arthur H. Blanchard, acting as Secretary; and present, also, 130 members and guests.

The second topic for discussion, "Cost Records and Reports," was opened by Nelson P. Lewis, M. Am. Soc. C. E., who was followed by Messrs. L. L. Tribus, James Owen, W. W. Crosby, G. W. Tillson, W. H. Connell, Henry W. Durham, Fred. E. Ellis, W. H. Messenger, Harry C. Hill, and A. M. Willcock.

Owing to the absence of Jean L. de Pulligny, M. Am. Soc. C. E., who was to have presented the opening discussion on the third topic, "Design of Highway Systems," the subject was introduced by Col. William D. Sohier.* The topic was also discussed by Messrs. Bertram Brewer, Nelson P. Lewis, James Owen, Amos Schaeffer, J. W. Howard, Ira O. Baker, C. E. Carter, Arthur H. Blanchard, and F. O. Whitney.

Adjourned.

January 18th, 1913.—The third special meeting was called to order at 10 A. M.; W. W. Crosby, M. Am. Soc. C. E., in the chair; Arthur H. Blanchard acting as Secretary; and present, also, 130 members and guests.

Francis P. Smith, M. Am. Soc. C. E., introduced the fourth topic, "Equipment for the Construction of Bituminous Surfaces and Bituminous Pavements." The subject was discussed further by Messrs. Henry B. Drowne, Clifford Richardson, Carl E. Pelz, E. H. Thomes, Charles P. Price, Arthur H. Blanchard, W. H. Kershaw, James L. Gaynor, Henry C. Poore, R. L. Christie, W. S. Godwin, Walter H. Fulweiler, J. A. Johnston, J. W. Howard, S. J. Stewart, W. W. Crosby, Philip P. Sharples, A. S. Brainard, and Prevost Hubbard.

Adjourned.

REGULAR MEETING.

February 5th, 1913.—The meeting was called to order at 8.30 P. M.; Vice-President J. Waldo Smith in the chair; Charles Warren Hunt, Secretary; and present, also, 94 members and 13 guests.

The minutes of the meetings of December 18th, 1912, and January 8th, 1913, were approved as printed in *Proceedings* for January, 1913.

A paper by B. F. Groat, Assoc. M. Am. Soc. C. E., entitled "Characteristics of Cup and Screw Current Meters; Performance of These Meters in Tail-Races and Large Mountain Streams; Statistical Synthesis of Discharge Curves," was presented by the author and illustrated with lantern slides.

The Secretary read communications on the subject from Messrs. W. G. Price, E. E. Haskell, and Charles H. Miller, and the paper was

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* Of the Massachusetts Highway Commission.

discussed orally by Messrs. J. C. Hoyt, C. W. Staniford, and the author.

A paper by John N. Brooks, Jun. Am. Soc. C. E., entitled "The Infiltration of Ground-Water into Sewers," was presented by title, but discussion was postponed until the next meeting of the Society, February 19th, 1913.

The Secretary announced the election of the following candidates on February 4th, 1913:

As Members

Joseph Marr Knight, Sedalia, Mo.
Walter Swain Nichols, Philadelphia, Pa.
Charles Lester Parmelee, New York City
William John Thomas, New York City
Max Wakeman Weir, Elba, N. Y.

As Associate Members

George Chester Britton, Hemlock, N. Y.
Charles Denton Conklin, Jr., Cheltenham, Pa.
Joseph Augustine Aloysius Connelly, New York City
Matthew Raymond Dinsmore, City of Mexico, Mexico
Louis Warren Duffee, Meridian, Miss.
Warren Clifford Earle, Portland, Ore.
Clarence Eugene Ellsworth, Washington, D. C.
Charles MacFarlane Finley, Sioux City, Iowa
Nicolay Knudtzon Fougner, Hongkong, China
Ernest Edmund Grimes, Juntura, Ore.
Thomas Felix Hickerson, Chapel Hill, N. C.
Alan Moore Edward Johnstone, New York City
Murray Kay, Hood River, Ore.
William Archibald Keene, Jr., Cotton Plant, Ark.
Fulton Lane, Los Angeles, Cal.
Ambrose Packard, Providence, R. I.
George Arthur Sampson, Brighton, Mass.
Harold Garfield Smith, Salt Lake City, Utah
Edward Smulski, Newton Highlands, Mass.
James Bigelow Steep, Indianapolis, Ind.
Henry William Taylor, Albany, N. Y.
Jent George Thorne, Clinton, Iowa
John Junior Wilson, Denver, Colo.

As Juniors

Clarence Myers Bates, Tacoma, Wash.
Frank Earle Dodge, Hudson, N. Y.
Edmond Anthony Fretz, Brooks, Alta., Canada
Johan Friedrich Wilhelm Gebhardt, Keokuk, Iowa

Peter Davidson Gunn Hamilton, Beverly, Mass.
Oliver Howard Lang, Atlanta, Ga.
Chester Sherman Lee, Cooperstown, N. Y.
Harold MacLean Lewis, Brooklyn, N. Y.
Maurice Paul O'Connor, New York City
John Fredrick Partridge, Stanford University, Cal.
Alvin Christian Rasmussen, Indianapolis, Ind.
William Stava, Coyote, Cal.
Harry Herman Steinhauser, Rochester, N. Y.
Percy Hixon Van Etten, Coyote, Cal.

The Secretary announced the transfer of the following candidates on February 4th, 1913:

From Associate Member to Member

Edward Everett Betts, Chattanooga, Tenn.
Harry Frank Cameron, Manila, Philippine Islands
Edward Walter Cunningham, Cleveland, Ohio
Wilton Joseph Darrow, New York City
Henry Roberts Hortenstine, Beaver Falls, Pa.
Jay Cowden Lathrop, Akron, Ohio
Ernest George Matheson, Coquitlam, B. C., Canada
Edward Emmet Sands, Calgary, Alta., Canada

From Junior to Associate Member

Ernest Delevan Cole, Concord, Cal.
Paul Max Entenmann, Brooklyn, N. Y.
Howard Foss Esten, Providence, R. I.
Henry James Gardner, Jr., Buffalo, N. Y.
Frank Gillelen, Los Angeles, Cal.
Frederick Nathaniel Hatch, New York City
Gordon Burnett Gifford Hull, Camargo, Chih., Mexico
Robert Louis Reimann-Hansen, Baltimore, Md.
Clarence Urling Smith, Chanhassen, Minn.
Charles Edwards Tirrell, Rochester, N. Y.
Royal Sylvester Webster, Havana, Cuba

From Junior to Associate

Alexander Allen MacVicar Russell, Oakland, Cal.

The Secretary announced the following deaths:

George Edward Gray, elected Member, July 2d, 1873; Honorary Member, June 5th, 1894; died January 1st, 1913.

William Gaston Hamilton, elected Member, October 7th, 1868; died January 23d, 1913.

Ebenezer Smith Wheeler, elected Member, November 7th, 1883; died January 5th, 1913.

Charles De La Plane Atterbury, elected Associate Member, November 8th, 1909; died June 3d, 1912.

John MacGregor, elected Associate, June 2d, 1903; died January 21st, 1913.

James Watson, elected Fellow, December 5th, 1872; died December, 1910.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract)

January 7th, 1913.—President Ockerson in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Clarke, Endicott, Gerber, Knap, Loomis, Macdonald, Ridgway, Snow, and Thomson.

A report* on a proposed amendment to the Constitution, increasing the number of Geographical Districts into which the Society is divided for the purpose of the nomination and election of representative officers, was adopted for presentation to the Annual Meeting on January 15th.

The following report was received from the Committee appointed to Recommend the Award of Prizes for the year ending July, 1912:

"The Committee appointed by you to recommend the award of prizes to be made at the Annual Meeting in January, 1913, submits the following nominations:

"For the Norman Medal, Paper No. 1212, 'The Detroit River Tunnel.'

"For the Rowland Prize, Paper No. 1210, 'The Reinforced Concrete Bridge Across the Almendares River, Havana, Cuba.'

"For the Collingwood Prize for Juniors, Paper No. 1213, 'A Reinforced Concrete Stand-Pipe.'

"Yours truly,
"E. P. Goodrich, *Chairman,*
"For the Committee."

On motion, duly seconded, the recommendations of the Committee were adopted, and the prizes were awarded as follows:

The Norman Medal to Paper No. 1212, "The Detroit River Tunnel," by Wilson Sherman Kinnear, M. Am. Soc. C. E.

The Thomas Fitch Rowland Prize to Paper No. 1210, "Reinforced Concrete Bridge Across the Almendares River, Havana, Cuba," by Eugene Klapp and W. J. Douglas, Members, Am. Soc. C. E.

The Collingwood Prize for Juniors to Paper No. 1213, "A Reinforced Concrete Stand-Pipe," by W. W. Clifford, Jun. Am. Soc. C. E.

The Secretary announced that the Special Committee appointed to Investigate the Condition of Employment of, and Compensa-

* See page 77.

tion of, Civil Engineers Throughout the Country, was now complete, all of the gentlemen appointed having accepted such appointment, and that the Committee is as follows: Alfred Noble, *Chairman*, S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, and J. A. Bensel.

The proposal for the appointment of a Special Committee "To Codify Present Practice on the Bearing Value of Soils for Foundations, and Report upon the Physical Characteristics of Soils in Their Relation to Engineering Structures," which was referred to the Board of Direction by the Society, was considered, and the Board decided to appoint such Committee.

The payment of $10 000 on the mortgage debt of the Society as soon as possible was authorized.

The purchase of twenty bonds, at a par value of $1 000 each, was authorized, said bonds to be added to the Reserve Fund already established.

Action was taken in regard to members in arrears for dues.

The resignations of 3 Members, 14 Associate Members, 1 Associate, and 8 Juniors, were accepted.

Ballots for membership were canvassed, resulting in the election of 9 Members, 25 Associate Members, 2 Associates, and 14 Juniors, and the transfer of 7 Juniors to the grade of Associate Member, and 1 Junior to the grade of Associate.

Ten Associate Members were transferred to the grade of Member.

Applications were considered, and other routine business transacted.

Adjourned.

January 15th, 1913.—The Board met, as required by the Constitution, at the House of the Society during the Annual Meeting, January 15th, 1913, at 1.05 P. M.; President Swain in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bates, Bensel, Bush, Cain, Churchill, Clarke, Connor, Edwards, Endicott, Hodge, Leonard, Macdonald, Metcalf, Ockerson, Ridgway, Smith, Snow, Staniford, Thomson, and Wallace.

The first business in order was the election of a Secretary.

Mr. Hunt retired.

Chas. Warren Hunt was nominated for Secretary and elected.

Mr. Hunt was recalled.

The following Standing Committees of the Board were appointed:

Finance Committee: Lincoln Bush, George C. Clarke, Henry W. Hodge, Leonard Metcalf, and Emil Gerber.

Publication Committee: James H. Edwards, Robert Ridgway, Chas. S. Churchill, William Cain, and Jonathan P. Snow.

Library Committee: J. Waldo Smith, Charles D. Marx, T. Kennard Thomson, E. C. Lewis, and Chas. Warren Hunt.

Adjourned.

February 4th, 1913.—President Swain in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Clarke, Edwards, Endicott, Hodge, Ridgway, Smith, Snow, Thomson, and Wallace.

It was resolved that no employee of the Society be permitted to accept fees or gratuities, and that a notice to this effect be conspicuously posted in the Society House.

It was resolved that all Reports of Special Committees intended for presentation to the Society at either of the two General Business Meetings shall be placed in the hands of the Secretary at least sixty days in advance of such meeting, and published in *Proceedings* in advance of their presentation for discussion.

The following Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, and report upon the Physical Characteristics of Soils, in their relation to Engineering Structures, was appointed: Robert A. Cummings, Edward C. Shankland, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner, and Frank M. Kerr.

A Committee of five members of the Board was appointed to consider and report upon an improvement of the methods of the presentation of papers read before this Society.

A Committee was appointed to draw up an amendment to the Constitution covering the proposed re-districting of the Society, on the lines indicated by the report of the Board to the Annual Meeting, and the discussion of the subject at that meeting.

Messrs. Charles D. Marx, William A. Cattell, Arthur L. Adams, and Charles Derleth, Jr., were appointed to serve, with the President and Secretary of the Society, as the representatives of this Society on a General Committee which is to have entire charge of the organization and carrying out of the proposed International Engineering Congress to be held in connection with the Panama-Pacific Exposition in 1915.

Ballots for membership were canvassed, resulting in the election of 5 Members, 23 Associate Members, and 14 Juniors, and the transfer of 11 Juniors to the grade of Associate Member, and 1 Junior to the grade of Associate.

Eight Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

REPORT IN FULL OF THE SIXTIETH ANNUAL MEETING, JANUARY 15TH AND 16TH, 1913.

Wednesday, January 15th, 1913 (10 A. M.)—John A. Ockerson, President, in the chair; Charles Warren Hunt, Secretary; and present, also, about 400 members.

Meeting called to order.

THE PRESIDENT.—The meeting will please come to order. This is the Sixtieth Annual Meeting of the American Society of Civil Engineers. The ballot for officers was closed at nine o'clock and the following tellers have been appointed: C. A. Tinker, W. F. Strouse, W. M. Kinney, E. G. Kastenhuber, Jr.

Tellers Appointed.

The next thing in order is the report of the Board of Direction.

Report of the Board of Direction.

The Secretary presented the Report of the Board of Direction.*

THE PRESIDENT.—If there are no objections, the report of the Board will be received and printed. There appear to be no objections. The report of the Secretary is next.

Report of the Secretary.

The Secretary presented his Report† of receipts and disbursements for the year, including a general balance sheet showing the financial condition of the Society.‡

THE PRESIDENT.—If there are no objections, the report of the Secretary will be received and printed as read.

Report of the Treasurer.

The report of the Treasurer.

JOSEPH M. KNAP, TREASURER, AM. SOC. C. E.—Mr. President and gentlemen, there is very little left for the Treasurer after the very full reports of the Secretary and the Board of Direction, but the Constitution requires that the Treasurer shall make a report at the end of the year, which is as follows:

The Treasurer read his report.§

Mr. President and gentlemen, I want to add a little to my usual formal report. In this, my last report as Treasurer of the American Society of Civil Engineers, I may be pardoned for elaborating the formal report by showing the steady and rapid increase in our cash, available assets, and cash investments, from the year 1900 to the present time. On December 31st, 1900, the balance sheet showed as follows:

Building and lots (cost)		$191 730.26
Due from members, etc		3 372.74
Cash on hand		9 929.82
Making a total of		$205 032.82
Deducting dues paid in advance	$9 528.03	
And mortgage deed at that time	75 000.00	
		84 528.03
We have		$120 504.79

* See *Proceedings*, Am. Soc. C. E., Vol. XXXIX, p. 10 (January, 1913).
† See *Proceedings*, Am. Soc. C. E., Vol. XXXIX, p. 18 (January, 1913).
‡ See *Proceedings*, Am. Soc. C. E., Vol. XXXIX, p. 17 (January, 1913).
§ See *Proceedings*, Am. Soc. C. E., Vol. XXXIX, p. 20 (January, 1913).

This is cash invested and on hand, and takes no account of any appreciation or depreciation of any real estate or property.

On December 31st, 1912, the corresponding items were:

Lots (cost)	$189 632.11	
House (cost)	170 955.59	
		$360 587.70
Invested in bonds		57 441.25
Due from members, etc		10 745.84
Cash on hand		44 074.04
		$472 848.83
Deducting dues in advance	$28 107.55	
Mortgage debt	115 000.00	
		143 107.55
Balance		$329 741.28

That is the amount of cash invested and on hand, but without any account of any appreciation in the value of the property. This is the amount actually earned, I might say, or saved during the twelve years, without taking into account the enhanced value of our property, which, as the report of the Board of Direction shows, has been very great.

Another point to which I would call your attention is the present mortgage of $115 000, which will be, or already has been, reduced to $105 000, and the reserve fund increased to $77 000. With the same rate of progress during the current year, the lines of our bonded debt and reserve fund will have crossed, so that the entire mortgage can be paid, leaving a cash balance in hand, during the present year. Whether that would be the best policy will be for the incoming Board to decide, but it would be indeed a great satisfaction for the Society to feel that all debts have been paid and an accumulation of a new reserve fund begun.

The President.—Gentlemen, you have heard the report of the Treasurer. If there are no objections, it will be received and printed as read. The next in order is the report on the award of prizes.

REPORT OF THE BOARD OF DIRECTION IN THE MATTER OF THE AWARD OF PRIZES FOR THE YEAR ENDING JULY, 1912.

The Committee appointed to Recommend the Award of Prizes consisted of Messrs. E. P. Goodrich, Gardner S. Williams, and J. W. Woermann. This Committee reported to the Board of Direction, and the Board, in accordance with the Constitution, has awarded the prizes in accordance with the recommendations of the Committee as follows: Award of Prizes.

The Norman Medal.—Paper No. 1212, "The Detroit River Tunnel," by Wilson Sherman Kinnear, M. Am. Soc. C. E.

Award of Prizes (continued).

The Thomas Fitch Rowland Prize.—Paper No. 1210, "Reinforced Concrete Bridge Across the Almendares River, Havana, Cuba," by Eugene Klapp and W. J. Douglas, Members, Am. Soc. C. E.

The Collingwood Prize for Juniors.—Paper No. 1213, "A Reinforced Concrete Stand-Pipe," by W. W. Clifford, Jun. Am. Soc. C. E.

By order of the Board of Direction,

CHAS. WARREN HUNT,
Secretary.

JANUARY 7TH, 1913.

Nominating Committee.

THE PRESIDENT.—The next in order is the appointment of the members of the Nominating Committee.

THE SECRETARY.—I beg leave to report a count of the final suggestions from members in the several districts expressing their choice for the member of the Nominating Committee to be appointed from each, as follows:

District No. 1: Total number of suggestions received, 379, as follows:

S. W. HOAG, JR.	187
RUDOLPH P. MILLER	139
S. H. WOODARD	51
GEORGE A. HARWOOD	1
C. V. V. POWERS	1

THE PRESIDENT.—Gentlemen, you have heard the names suggested for member of the Nominating Committee for District No. 1. What is your pleasure?

G. W. TILLSON, M. AM. SOC. C. E.—Mr. President, I move that as Mr. S. W. Hoag, Jr., has received the largest number of votes, he be declared member of the Nominating Committee from District No. 1.

(Motion duly seconded.)

THE PRESIDENT.—It is moved and seconded that S. W. Hoag, Jr., be the member of the Nominating Committee from the First District. As many as are in favor of that will signify it by saying "aye"; contrary, "no". Carried.

THE SECRETARY.—District No. 2: Total number of suggestions received, 169, as follows:

CHARLES T. MAIN	82
C. M. SPOFFORD	64
JOHN E. HILL	18
J. W. ROLLINS, JR.	2
A. C. DENNIS	1
JOHN KENNEDY	1
H. N. RUTTAN	1

THE PRESIDENT.—What is your pleasure, gentlemen?

A MEMBER.—Mr. President, I move that Charles T. Main, having received the largest number of votes, be made the member for District No. 2.

(Motion duly seconded.)

THE PRESIDENT.—It is moved and seconded that Charles T. Main be selected as member of the Nominating Committee from District No. 2. As many as are in favor of that will signify it by saying "aye"; contrary, "no". It is so ordered.

THE SECRETARY.—District No. 3: Total number of suggestions received, 202, as follows:

E. A. FISHER	81
ENRIQUE A. TOUCEDA	62
WILLIAM H. YATES	56
GEORGE H. FENKELL	1
LOUIS H. KNAPP	1
MORRIS R. SHERRERD	1

G. S. WILLIAMS, M. AM. SOC. C. E.—Mr. President, I move that E. A. Fisher, having received the highest number of votes for District No. 3, be declared the member of the Nominating Committee from that district.

(Motion duly seconded.)

THE PRESIDENT.—It is moved and seconded that E. A. Fisher be declared the member of the Nominating Committee for District No. 3. As many as are in favor of that will signify by saying "aye"; contrary, "no". Carried.

THE SECRETARY.—District No. 4: Total number of suggestions received, 265, as follows:

RICHARD KHUEN	151
FRANK SUTTON	65
E. K. MORSE	47
RICHARD L. HUMPHREY	1
MORRIS KNOWLES	1

N. P. LEWIS, M. AM. SOC. C. E.—Mr. President, I move that Richard Khuen, having received the highest number of votes, be declared member of the Nominating Committee for District No. 4.

(Motion duly seconded.)

THE PRESIDENT.—Gentlemen, it has been moved and seconded that Richard Khuen be declared member of the Nominating Committee for District No. 4. All in favor of the motion signify by saying "aye"; contrary, "no". It is so ordered.

Nominating Committee (continued).

THE SECRETARY.—District No. 5: Total number of suggestions received, 181, as follows:

J. B. BERRY	68
J. V. HANNA	58
J. M. JOHNSON	28
E. H. LEE	25
WILLIAM G. RAYMOND	1
EDWARD E. WALL	1

MR. G. S. WILLIAMS.—Mr. President, I move that J. B. Berry, having received the highest number of votes, be declared the nominee for District No. 5.

(Motion duly seconded.)

THE PRESIDENT.—It has been moved and seconded that J. B. Berry be selected as member of the Nominating Committee for District No. 5. As many as are in favor signify by saying "aye"; contrary, "no". It is so ordered.

THE SECRETARY.—District No. 6: Total number of suggestions received, 180, as follows:

FRANK M. KERR	63
T. U. TAYLOR	41
B. M. HALL	40
J. F. COLEMAN	24
CHARLES C. WENTWORTH	2
M. J. CAPLES	1
LOUIS C. HILL	1
E. H. LEE*	1
RICHARD MONTFORT	1
MAURY NICHOLSON	1
ARSÈNE PERRILLIAT	1
ARTHUR PEW	1
J. S. SEWELL	1
G. R. SOLOMON	1
NISBET WINGFIELD	1

THE PRESIDENT.—What is your pleasure, gentlemen?

MR. N. P. LEWIS.—Mr. President, I move that Frank M. Kerr, having received the largest number of votes, be declared member of the Nominating Committee for District No. 6.

(Motion duly seconded.)

THE PRESIDENT.—It is moved and seconded that Frank M. Kerr be declared a member of the Nominating Committee for District No. 6. As many as are in favor signify by saying "aye"; contrary, "no". It is so ordered.

*Mr. Lee is ineligible, as he does not reside in this district.

THE SECRETARY.—District No. 7: Total number of suggestions received, 298, as follows:

WILLIAM MULHOLLAND	147
D. D. CLARKE	89
H. N. SAVAGE	52
J. D. GALLOWAY	1
SAMUEL H. HEDGES	1
D. C. HENNY	1
F. C. HORN	1
F. H. JOYNER	1
J. B. LIPPINCOTT	1
CHARLES D. MARX	1
EMERY OLIVER	1
W. J. ROBERTS	1
A. J. WILEY	1

THE PRESIDENT.—Gentlemen, William Mulholland apparently has received the largest number of votes. As many as are in favor of declaring Mr. Mulholland the member of the Nominating Committee for District No. 7 signify by saying "aye"; contrary, "no". It is so ordered.

Report of Committee on Concrete and Reinforced Concrete.

The next in order is the report of the Special Committee on Concrete and Reinforced Concrete. This paper is too long to read in detail. I shall ask some member of that Committee to state to the Society the general features of that report.

RICHARD L. HUMPHREY, M. AM. SOC. C. E.—Mr. President, the work of the Committee since its last report of January, 1909, is best expressed in the paragraphs of the report which I will read.*

J. P. SNOW, M. AM. SOC. C. E.—I move, Mr. President, that the report be received.

(Duly seconded.)

THE PRESIDENT.—It has been moved and seconded that the report just made be received and the Committee continued. As many as are in favor of the motion signify by saying "aye"; contrary, "no". It is so ordered.

Report of Committee on Engineering Education.

The next in order is the report of the Special Committee on Engineering Education.

DESMOND FITZGERALD, PAST-PRESIDENT, AM. SOC. C. E.—Mr. President, I have a report, signed and in proper form, which I carefully laid aside and came off and left behind me. It only shows what remarkable things a man can do; but, on the other hand, perhaps it is just as well, Mr. President, that that is the case, because almost all formal reports are more or less colorless. There is very little warmth

* See p. 117.

Report of Committee on Engineering Education (continued).

in them; and perhaps I can give a little more by stating briefly what the Committee has done.

At the last meeting, or the meeting before, I think I showed you some diagrams that we had made in regard to the studies pursued in the different colleges and technical schools of the country. After the Committee had dipped into that subject for about two years they made up their minds that it was a thing that we should have a large appropriation to continue properly; and the American Society of Civil Engineers at that time being rather poor, and with no available funds, we could not undertake the thing with a large force. We could only do what we could with individual efforts of the members of the Committee.

Now, I suppose there is no bigger subject to run up against than engineering education, and you know when you run up against a big subject it takes a lot of time, and you have to be very careful not to get caught by the enemy. It is just the same as with a fortress, the bigger the fortress the more you have to attack it from all sides and the longer it takes. The great thing, Mr. President, seems to be not to make any mistakes, and it was a rich field for pitfalls.

Now, the Carnegie Foundation, which is very liberally endowed through Mr. Carnegie's thoughtfulness, certainly is in a position where it can make this investigation and gather all the data, for twenty or forty thousand dollars, or whatever amount it may cost to do it, in good form and properly, and it has agreed to do it. It is simply a question of time. Since our last meeting, since I made my last report, Dr. Pritchett has been very ill for nearly a year, has been away in California, and nothing could be done. Now, I should like to say, for the information of those who do not know about our Committee on Engineering Education, and I say this because I have heard that some people think we have not done any work, but I think they are the people who have not done much work themselves in this direction, that we have done a good deal of work, and we have collected a good many data.

We have worked in connection with a large committee, a very large committee, and I should like to read the names of the committee, if we have a little time. Before beginning, I want to say that during the last year we have lost one of the most valuable members of our Committee by the death of Major B. M. Harrod, of New Orleans, and I need not say a single word in regard to that lamentable occurrence. I do not think the American Society of Civil Engineers ever had a warmer friend or a finer engineer than Mr. Harrod. Personally, I relied a good deal on his judgment and his great interest. On this subject of Engineering Education he wrote me long letters, and we had frequent correspondence about it, and there was no kinder or nobler man than Mr. Harrod.

With Mr. Harrod was Mr. Bates and Professor Mead and myself, representing the American Society of Civil Engineers; Messrs. Frederick W. Taylor, of Philadelphia, and Alexander C. Humphreys, of Stevens Institute, representing the American Society of Mechanical Engineers; from the Electrical Engineers, Mr. Charles F. Scott and Dr. Samuel Sheldon, of the Polytechnic Institute of Brooklyn; from the American Chemical Society, Mr. Clifford Richardson, of New York, and Professor H. P. Talbot, of the Massachusetts Institute of Technology; from the Mining Engineers, Dr. Henry M. Howe, of New York, and Dr. John Hays Hammond, also of New York. Then came Dean James M. White, of Urbana, Ill., Professor C. L. Crandall, of Ithaca, N. Y., Professor D. C. Jackson, of the Massachusetts Institute of Technology, and Dr. Henry S. Pritchett, of the Carnegie Foundation, and he was the last addition to the Committee.

Now, gentlemen, I do not know that the Committee itself can carry the work any further than it has done. I think that our usefulness, after having brought about the appropriation of the necessary funds by the Carnegie Institute to carry on this work—and it will probably take three or four years to finish it, and they are willing to spend a liberal sum of money—I think our work is practically ended. At the same time, it is very necessary, perhaps, and I for one shall continue to work in this direction, even if I am not a member of a committee. I do not know whether to recommend the continuance of the Committee, or that it be discharged, but I should think that it might be continued for a year to enable Professor Mead and Mr. Bates to aid in giving any advice as to the steps to be taken in the future. Thank you very much.*

The President.—It seems to me that it is rather important that this Committee should be continued, to give direction to the work that Professor Pritchett proposes to do, and, if there are no objections, the report will be received and the Committee continued.

Report of Committee on Columns and Struts.

The next in order is the report of the Special Committee on Steel Columns and Struts, Austin L. Bowman, Chairman.

Mr. Bowman presented a Progress Report of the Special Committee on Steel Columns and Struts.†

The President.—Gentlemen, you have heard the Progress Report of the Special Committee on Steel Columns and Struts, and the same action will be taken in this case. The report will be received and the Committee continued. The next in order is the report of the Special Committee on Bituminous Materials for Road Construction, W. W. Crosby, Chairman.

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* The Progress Report of the Special Committee on Engineering Education, received subsequently, is printed on page 169.

† See p. 170

Report of Committee on Bituminous Materials for Road Construction.

Mr. A. H. Blanchard presented a Progress Report of the Special Committee on Bituminous Materials for Road Construction.*

The President.—Gentlemen, if there are no objections, the Progress Report of the Special Committee on Bituminous Materials for Road Construction will be received and the Committee continued.

Report of Committee on Public Utilities.

The next in order is the Progress Report of the Special Committee on Public Utilities, Mr. Frederic P. Stearns, Chairman.

Mr. Stearns presented the following report:

PROGRESS REPORT OF SPECIAL COMMITTEE ON VALUATION OF PUBLIC UTILITIES.

To the American Society of Civil Engineers:

The Committee to Formulate Principles and Methods for the Valuation of Railroads and other Public Utilities is pleased to be able to report substantial progress.

Careful search has been made in the Library of the Society and bibliography prepared of articles and references to the subject under investigation.

The effort has been made to get in touch with the work of the various public service commissions and to obtain copies of their reports.

A good beginning has been made in the study of and report upon the principles underlying the art of valuing public utility properties.

Your Committee asks to be continued.

Respectfully submitted,

Frederic P. Stearns, *Chairman,*
For the Committee.

January 14th, 1913.

The President.—The Progress Report of the Committee will be received and the Committee continued. The next in order are reports from the Board of Direction.

Special Committee on Employment and Compensation of Civil Engineers.

The Secretary.—I have to report, Mr. President, that the Board of Direction has appointed a Special Committee to Investigate the Condition of Employment of, and Compensation of, Civil Engineers throughout the country, in accordance with the action of the Society, which referred the appointment of that Committee to the Board. The Committee is Alfred Noble, Chairman, S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, and John A. Bensel.

Special Committee on Bearing Value of Soils.

I also have to report that the Board of Direction has decided to appoint a Special Committee of seven to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., but as yet the Committee has not been completed.

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* See p. 175.

TO THE MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS IN ANNUAL MEETING ASSEMBLED, JANUARY 15TH, 1913:

Report of Board of Direction on Geographical Districts.

For a number of years the territory occupied by the membership of the Society has been divided into seven Geographical Districts, of which District No. 1 is the territory within 50 miles of the Post Office of the City of New York, and each of the other six, as nearly as practicable, contiguous territory, each containing, as nearly as practicable, an equal number of members.

It has always been a very difficult matter to arrange the Districts so that each of them should be in any sense local. In the Districts as now laid out, for instance, West Virginia, Arizona, and New Mexico, are in the same District; Kentucky and North Dakota are in the same District; Colorado and Washington are in the same District, and District No. 2 takes in the whole of the Dominion of Canada and the New England States, together with Europe and Africa.

As the membership increases it becomes occasionally necessary, under the Constitution, in order that the Districts shall be arranged "as nearly as practicable" with the same number of members in each, to make a re-districting, and during the past six months the Board has been endeavoring to do this. The result has not been particularly satisfactory, geographically, the only gain over the present division being a little more equal distribution in the number of members. For instance, in the best plan that has been worked out, Indiana, North Carolina, Florida, Texas, and New Mexico, are all in the same District. Illinois is in the District with Idaho and Montana, and New York State is in the same District with Minnesota. The second District is practically as it is at the present time, containing the whole of Canada, the New England States, and Europe.

It has been suggested that the representative character of the members of the Board of Direction could be better maintained if the territory outside of the Resident District was divided into twelve instead of six Districts as at present, making thirteen Districts in all. The number of Directors would in this case not be changed, and each of the twelve Districts would have at all times one Director, the number of representatives from District No. 1 remaining unchanged.

In the same way, instead of the appointment of two representatives from each of the Geographical Districts on the Nominating Committee, one representative would be elected from each (except from District No. 1), so that the Nominating Committee would be made up of two representatives of District No. 1, and one from each of the other twelve Districts.

With the present membership of the Society, there are about 1 400 Resident Members, and including foreign countries, the remainder of the membership is about 5 400. If this latter number were divided into twelve Districts there would be at the present time about 450

Report of Board of Direction on Geographical Districts (continued).

in each of the Districts. It is believed that a sub-division into twelve Districts can be worked out on State and Territorial lines, which would not only be more representative of certain sections of the country, but would also be much more permanent than is possible with only six Districts.

It is not possible to make this change except by an amendment to the Constitution, and the Board recommends to the Annual Meeting that the whole matter be taken up for discussion.

Under the Constitution, proposed amendments must be reduced to writing, signed by not less than five Corporate Members, and be presented to the Secretary not less than sixty days previous to the Annual Convention. Such amendments are then sent by letter to all Corporate Members of the Society at least twenty-five days previous to the Annual Convention, and are in order for discussion at the Business Meeting held during the Annual Convention. They may be amended in any manner pertinent to the original amendment by a majority vote of the Business Meeting, and if so amended shall be voted upon later by letter-ballot; if not so amended, they shall be voted upon by letter-ballot, as submitted.

If this course is taken in this matter, and an amendment to the Constitution covering this suggested change adopted, it will go into effect November 1st, 1913, in ample time for a change of the Districts before 1914.

The necessary changes to be made are very simple, and are as follows:

Amend Article VII—Nomination and Election of Officers—as follows:

Section 1, second line, strike out the word "seven", and substitute the word "thirteen".

Line 8, strike out the word "and" between 6 and 7, and add ", 8, 9, 10, 11, 12, and 13."

Strike out the first paragraph of Section 2, and substitute the following: "At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, shall be appointed by the meeting to serve for two years. They shall be selected so as to provide, with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve districts; and these with the five living last Past-Presidents of the Society, shall be a Committee to nominate officers for the Society."

Strike out the word "and" in the 15th line of the third paragraph of Section 2, and after the figure 7, add ", 8, 9, 10, 11, 12, and 13."

The whole article as it will read when so amended is attached to this report.

By order of the Board of Direction,

CHAS. WARREN HUNT,
Secretary.

JANUARY 7TH, 1913.

Discussion on Geographical Districts.

THE PRESIDENT.—Gentlemen, the recommendation of the Board in regard to districts is before you. What is your pleasure?

THE SECRETARY.—Mr. President, I might say that the intention of the Board, in bringing this matter before the Annual Meeting, was to get an expression of opinion, if possible, and to have some discussion of this suggested change, before it is placed before the Society in the form of an Amendment to the Constitution. In this way, the opinions expressed here will come before all the members in the *Proceedings*.

MR. G. W. TILLSON.—Mr. President, in order to get the matter before the meeting, I move that the recommendation of the Board be approved.

(Motion seconded.)

THE PRESIDENT.—Gentlemen, you have heard the motion moved and seconded. As many as are in favor of the motion signify it by saying "aye"; contrary, "no"; the recommendations of the Board are approved.

S. WHINERY, M. AM. SOC. C. E.—Mr. President, I understood that the object of this motion was to open this question to discussion by the meeting, and I think that that should be done. If necessary, therefore, I move that that motion be reconsidered.

(Motion duly seconded.)

MR. R. L. HUMPHREY.—What motion?

THE PRESIDENT.—I suppose we can go on with the discussion, if the gentlemen do not object to reconsidering it, without any further ceremony.

MR. R. L. HUMPHREY.—Do I understand that this meeting does not accept a recommendation of the Board to send out an amendment without being sent out sixty days in advance by letter-ballot? Why cannot this meeting order such an amendment to ballot, and have it sent out?

THE SECRETARY.—Such action would not be constitutional.

MR. R. L. HUMPHREY.—Mr. President, I submit that if this Annual Meeting should order this sent out unanimously, it would be perfectly legal.

THE PRESIDENT.—It cannot go out through letter-ballot until it has been presented to the Annual Meeting; as I understand it, the object of presenting it here is for the purpose of getting an expression of opinion from this meeting, and the expression that we give here will go out to all the members of the Society, so that they can act intelligently on the matter.

H. M. WILSON, M. AM. SOC. C. E.—Mr. President, I think I see the point that Mr. Humphrey intended to raise when he made his inquiry. I realize, as the Secretary states, that it would be unconstitutional to send this amendment out to letter-ballot at this time; but, for the purpose of getting a little discussion here, which it seems the Board would like to have, although it seems doubtful that you will get much, because I judge that, from their actions, most of the members here approve of

Discussion on Geographical Districts (continued).

this recommendation, I make this suggestion: The next action, as I understand, is the presentation in writing, by five members, of an amendment to the Constitution, and I make the inquiry as to whether or not, for the sake of debate here at this well attended meeting, five members might not get on their feet and say, "I will sign such a resolution as has just been presented." If that would meet the wishes of the Society and the President in the matter of opening the debate and starting the ball rolling, I may say it will give me great pleasure to sign such a resolution as has been offered.

Mr. G. S. Williams.—Mr. President, I fully appreciate the sentiments of the Board of Direction in proposing these amendments, or this amendment; and, as far as it goes, it is all right, but it does not go far enough. Dividing six districts in two and making twelve instead of six, will not increase the number of members of the Board of Direction in attendance upon its meetings.

This Society has gotten so large now, we are reaching all over the world, and those who are on the outskirts, on the Pacific Coast, even in the Mississippi Valley, nevertheless feel somehow or other that they do not have enough to say about the running of it. I want to tell them—and I am one of that group, because I belong out in the central portion of the country—that they have as much to say about the running of it as they are willing to take upon themselves.

The reason the Pacific Slope does not run the American Society of Civil Engineers is because it does not get around here and do it; and the reason the Southern States do not run it is the same; and the reason that the Central States do not run it is the same; and the reason that the members here in New York and a few from New England and a few from Philadelphia do run it is because there is not anybody else around to do it, and the Society has to be run.

Now, I hear more or less sometimes from my friends in the outlying districts that this, that, and the other fellow is running the American Society of Civil Engineers. All I have to say to you who are complaining is, "get busy and run it yourselves". What we want here is not more representation or more diversified representation, but something that will get the fellows who are supposed to represent the business of this Society down here to do business, and I think that this Society is big enough and strong enough and wealthy enough to make it an object for each member of the Board of Direction to get here and do the business of the Society.

Now, while I am in favor of this amendment as far as it goes, I think that the Board of Direction or some specially constituted committee of this Society should carefully take into consideration a project to make our members on the Board of Direction from the outlying districts something more than mere figureheads.

The President.—Gentlemen, I might give some word of my experience, and that is that at every Board meeting that I have attended here there have been members from Boston and from Roanoke and from Chicago, and various sections of the country, and there is no reason, as far as the authority of the Board is concerned, why the rest of the members of the Board should not have been present.

Mr. N. P. Lewis.—Mr. President, it seems to me that the thing we are most interested in is how these new men will affect the membership of the Nominating Committee. That Nominating Committee, I believe, consists of nineteen, four members elected from each district, and five past-presidents. Under the present system, every year there are eight of the nineteen new members elected to the Committee, not quite a majority. Under the new system there would be fourteen out of the nineteen new members elected every year, if I understand it.

The Secretary.—I am afraid you do not understand it.

Mr. N. P. Lewis.—Twelve?

The Secretary.—Six.

Mr. N. P. Lewis.—Twelve new members from each of the twelve new districts.

The Secretary.—Six.

Mr. N. P. Lewis.—As I understand the report of the Board of Direction, it was that there be twelve districts outside of District No. 1, with one member elected from each of those districts each year.

The Secretary.—One member from six of the districts to be elected every year.

Mr. N. P. Lewis.—Every two years?

The Secretary.—One member from the six alternate districts to be elected the year thereafter.

Mr. N. P. Lewis.—Then the changes in the personnel would be the same as they are at present?

The Secretary.—Yes, sir.

Mr. N. P. Lewis.—The only other thing I want to speak about is this: Mr. Williams, I think, intended—possibly I misunderstood him—to refer to members of the different districts not attending the meetings of the Nominating Committee or of the Board of Direction?

Mr. G. S. Williams.—It applies to both. I was speaking of the Board of Direction more particularly.

Mr. N. P. Lewis.—It is true that it is an exceedingly difficult thing, and would likely be more difficult to have members of twelve outlying districts rounded up at the meeting of the Nominating Committee. This may not be germane to this particular amendment that has been proposed, but it might be possible, and it seems to me not unfair, to have the meeting of the Nominating Committee held in the different districts, provided—and I think this is only fair—provided that the expenses of the members of the Nominating Committee be paid for

Discussion on Geographical Districts (continued).

attendance of the members at that meeting. I believe there is a provision for that at the present time.

If you want to get local color, local atmosphere, let us have the meetings of the Nominating Committee held in different districts in successive years. It is proper that there should be some sort of continuity of purpose or quality in that committee. It is wise to have a certain number of holdovers unquestionably every year, and I cannot but believe that twelve districts, outside of the home district, would add interest to the meetings of the Committee, and if the meetings of the Committee could perhaps alternate every second year in the home district, and in other years in the various other districts in rotation, it would infuse a little more interest and life into the work of the Nominating Committee.

H. B. Seaman, M. Am. Soc. C. E.—Mr. President, in connection with the suggestion that the expenses of the Nominating Committee be paid for their attendance on the meetings, I believe that there is, or there has been, no provision by which their expenses are paid except by special action of the Board of Direction. I understand that for years past it has been found advisable to pay the expenses of directors who are willing to give their time to come to New York City to help manage the Society, but there has been no provision by which the Nominating Committee, which is the father of the Board of Direction, has its expenses paid. There are a number of men who went from the East to Seattle last summer, with no such provision. They paid their own expenses, and they gave their own time, and some of them had very little time to give. I know of one individual who went from Philadelphia to Seattle and spent half a day there to do his work, and came home. There is no provision in the Constitution to pay expenses, the actual expenses of such men.

There was a motion put in the Convention at Seattle that their expenses be paid, the same as those of the members of the Board are paid. I would like to ask, on behalf of the membership of the Society, what has been done by the Board in regard to that motion, if those expenses have been paid?

The Secretary.—Mr. President, the Board of Direction considered the resolution that was adopted at the Annual Convention in Seattle, and adopted a resolution to pay mileage to the members of the Nominating Committee at any one meeting that they should decide upon, whether it should be at the place of holding the Annual Convention or at some other point which would be more convenient for them; and that will be done in the future.

Mr. H. B. Seaman.—Was it done for those who went to Seattle?

The Secretary.—It was not, sir.

Mr. H. B. Seaman.—There you have the situation, gentlemen. We have a modification of the Constitution proposed by which if ten

men do not attend the meetings it would revert to the Board of Direction, which would practically make the Board of Direction a self-perpetuating body, because no men would ever be got together across country to attend that meeting. I do not think there is a record of any ten members of that committee attending the meeting. It could all be done by correspondence, but the provision of the Constitution is that they must attend. If they do not attend, it would revert to the Board of Direction, and they would become a self-perpetuating body, and it was understood by the Society that that would be attended to by the Board itself. I think the key to this situation is to nominate a Nominating Committee which can nominate and be ready to attend.

Mr. R. L. Humphrey.—Mr. President, in District No. 4, we have Maryland and Washington and the District of Columbia, Delaware, Pennsylvania, and Ohio. Now, in the vicinity of Pennsylvania those who live in the southern end of Virginia and do their business in that particular section want to vote in that section, District No. 4. I think the districts should be fixed with some idea as to the centers of population in the districts selected. I think that District No. 4 should comprise the State of Pennsylvania and all of New Jersey which is not in the first District. I think this is a matter in which there should be action taken. It is manifestly impossible for the people in Ohio to be in touch with the people of Pennsylvania, in attempting to get a proper representation of the Nominating Committee, and it seems to me that the provision dividing the districts into twelve is a proper one, and it is a pity that the red tape of this Society prevents us from taking needed action on something which we think is necessary.

Mr. J. P. Snow.—Mr. President, the point made by the last speaker in attempting to fix upon a new redistricting should be taken into consideration, but our last amendment to the Constitution precluded the dividing up of a State. I agree with Mr. Humphrey that that ought to be borne in mind, because in making these districts the prominent cities are the points to be considered with their contiguous territories. Philadelphia and Pittsburgh are in the same State, and it is a big one, and in order to make a good distribution, we need to divide some of the States, make the districts other than on Territorial or State lines; but, as I understand the resolution read by the Secretary, it simply states that it is thought that such a division can be made. It does not preclude the dividing of a State.

I think that the last amendment to the Constitution should be somehow abrogated by amendment, so that a State can be divided.

There is no particular difficulty in doing it, because very frequently we find that in the central portion of a State, like New York or Pennsylvania, there are very few of our members resident, and a line

Discussion on Geographical Districts (continued).

could be drawn which would make much better work, much better distribution, than on State lines.

The President.—I want to say one word. The report does not contemplate lines on which districts shall be drawn; that is left to the Board of Direction, and I think you can rely on them to make the distribution so as to have certain large centers in each of the districts.

That will be the aim, as I understand it.

A. N. Talbot, M. Am. Soc. C. E.—Mr. President, the provision for increasing the number of districts for the Nominating Committee strikes me as being a good one, but the other provision that a director shall be selected from each of those districts may lead us to have some directors from these far-away districts, and in this way increase the difficulty of getting a general attendance at the meetings of the Board of Direction. It seems to me that this part of the proposal should be considered further before being put on as a change in the Constitution, unless in some way the number of meetings of the Board could be reduced from monthly meetings to say quarterly meetings, in which case greater efforts could be made than at present.

James Burden, M. Am. Soc. C. E.—The membership of the Society is so great that it seems to me it might be well, not only to increase the number of districts, but to increase the number of men representing those districts. Would it not be well to increase the number of districts and leave its membership at two members as it is now?

Mr. H. M. Wilson.—Mr. President, the suggestion of the last speaker, which I note meets with some favor, opens up, even further than do Mr. Williams' remarks, the immensity of this question and its importance to the Society. I agree with Mr. Williams; I should go farther in an endeavor to have the meetings of the Society well attended; and I have cast rather a longing eye on the growing annual surplus of $30 000. I feel hungry, for one thing, physically hungry, because I understand that a resolution which was referred to the Board last year, suggesting that an effort be made to entertain us with luncheon in this building, so that we could come here and discuss these matters conversationally has been turned down. (Applause.) I presume that applause is not caused by agreement with what I understood was the opinion of a member of the Board of Direction, that it would be undignified for us to eat our luncheon here in this manner without cost. Therefore, I should be glad for one, to use some of the surplus for other purposes than furnishing luncheon, provided the members of the Board of Direction will permit us to pay for the luncheon here next time. Would it not be to the interest of this Society to pay, not only the traveling expenses of the Board of Direction, when they attend the Annual Meetings, but to pay them salaries as in some cor-

porations? Is not the business of this Society large enough to warrant that?

In making that suggestion, Mr. President, I do not say that I wholly agree to the proposition myself at this time, but it occurred to me as a thought worthy of consideration, while we are here and are debating this subject, as to whether or not we should so increase the membership of the Board of Direction as to make it prohibitive that we pay them any compensation for attendance, and perhaps even prohibitive that we even go so far as to pay traveling expenses for them and for the members of the Nominating Committee.

Mr. S. Whinery.—Mr. President and gentlemen of the Society, I think that a more important and interesting question is whether this provision of the Constitution and the practice of the Society, dividing the country into geographical districts, might not be advantageously abandoned altogether. Aside from one or two small items of the language proposed, I doubt if in the actual practice of administering the affairs of this Society, this question of geographical location enters in any important way. It may be remarked that our Society is, if I am correctly informed, the only large engineering or technical society in this country in which this geographical distribution is maintained.

It has been found entirely practicable and satisfactory to the other societies to do without such a geographical division. It seems to me the question might be very profitably considered here, whether this Society might not do away entirely with that whole provision and all the consequences and difficulties and complications that attach to it.

If, however, it is maintained, it seems to me that, with regard to the outlying districts, it is not necessary, under the Constitution or in practice, so far as I understand it, that the person elected as a Director for that district shall be resident in the district. Am I right about that, Mr. President, that Directors elected must be resident in the district?

The Secretary.—At the time of his nomination, the Constitution says he must be resident there, and if elected he continues to represent that district, although he may move away.

Mr. S. Whinery.—He may move away; he may become a non-resident of that district. Now, why would it not be practicable and satisfactory to adopt practically the same idea that they pursue in Parliament in England? District No. 7, which is scattered, could take in members here in New York to attend all the meetings without difficulty, and they could be made to represent the interests of that district, so far as such representation is of any consequence. I doubt very much if it is of any importance, and, as a matter of fact, I think the affairs of the Society would probably be administered just as advantageously and just as satisfactorily by a body of directors elected

Discussion on Geographical Districts (continued).

within reasonable distances of New York City, as under the present system; but I think the main question that should be considered is whether it would not be wise to dispense entirely with this system of geographical distribution.

E. W. Stern, M. Am. Soc. C. E.—Mr. President, I should like to say a word on this question in relation to the method we now have of appointing the Nominating Committee. Inasmuch as the Nominating Committee is probably the most important body which has to do with this organization, inasmuch as it nominates the Board of Direction, I do believe that the policy or the system of having the Nominating Committee elected by informal ballot, in the manner at the present time in use, is not one which would get probably the best results. I believe that, inasmuch as the members of the Board of Direction are elected by the Australian ballot system, that the Nominating Committee should likewise be elected by the Australian ballot system. I see no reason why it should be otherwise.

We nominate these men, give them tremendous powers, and they nominate a Board of Direction, which in turn is elected by the Australian ballot system, but on very few occasions has the recommendation of this Nominating Committee ever been changed in the final balloting. We could really logically have an informal ballot on the recommendation of the Nominating Committee rather than an informal ballot on the election of the members of this Nominating Committee, which is the custom in vogue to-day.

The President.—The question before the meeting is the report of the Board of Direction in regard to the proposed amendment, and we are not discussing the methods of selecting the Nominating Committee, or things of that sort. The discussion has been rather rambling.

Mr. E. W. Stern.—With all due deference to your opinion, Mr. President, it seems to me that it has a bearing on the whole situation, as involving the methods of election of the Board of Direction and the subdivision, and so on. However, I defer to your judgment.

Mr. R. L. Humphrey.—In order to get a motion before the meeting, I should like to move, Mr. President, that it is the sense of this meeting that such a change as outlined by the report read by the Secretary is desirable, that the Board, in reporting back such an amendment to the Society, remove the restrictions as to State and Territorial lines.

The President.—Gentlemen, you have heard the motion. All in favor signify by saying "aye"; contrary, "no". The ayes seem to have it; the ayes have it.

Mr. G. W. Tillson.—Mr. President, is the original motion before the meeting? It seems to me, from the discussion we have had, that there is no question at all about adopting the report of the Board in general. It seems to me that if we should adopt that and then carry some other motion, which would make certain suggestions, that would

be the better way to proceed, either that the proposed amendment be drawn up by the Board or by the committee, whichever is seen fit. It seems to me, first, that we need to act upon the recommendation of the Board as a thing by itself.

Mr. R. L. Humphrey.—Mr. President, I think the motion offered by myself covered that. It approved the suggestion and reported back the amendment that the restrictions should be removed.

The Secretary.—Mr. President, an amendment to the Constitution may be proposed by any five members of the Society. The object of the Board, in making this report to this Annual Meeting, was to have a discussion in order that the ideas of the members here could be gotten. Now, I think the Board does not need any further instructions about the matter, unless this meeting wants to pass a resolution that it does not approve of any part of this proposition. The suggestion that was made about the removal of the State and Territorial lines is certainly a very good one. That, as far as I have heard, is about the only suggestion that has been made for a change.

Mr. S. Whinery.—Mr. President, in order to get the trend of this meeting with reference to the question of geographical distribution, I move that it be the sense of this meeting that the provision for the geographical distribution of members and every other item of the Constitution which looks to geographical distribution be changed and abolished.

The President.—It seems to me that we have already approved of the recommendation of the Board.

Mr. S. Whinery.—I intend to introduce this as an entirely new motion. I understood your decision to be that the other question was disposed of.

A Member.—Is there a motion before the house?

The President.—There is no motion.

A Member.—I second the motion.

The President.—Will you state that motion again?

Mr. S. Whinery.—That motion is that it be the sense of this meeting that all provisions in the Constitution relating to the division of the territory into geographical sections and to sectional representation and everything else relating to that matter be abolished.

The President.—Gentlemen, are you ready for the question?

Mr. G. S. Williams.—I believe that Mr. Humphrey's motion was seconded. Was it ruled out of order by the chair?

The President.—No; it was passed, carried.

Mr. G. W. Tillson.—Mr. President, just for information, I would like to know what became of my motion?

The President.—Your original motion was carried, but there was a request afterward that the subject be discussed, and I was very glad to have it discussed; and Mr. Humphrey's motion, as I under-

Discussion on Geographical Districts (continued).

stand it, is practically identical with yours. Now, this motion by Mr. Whinery is before the house. Are you ready for the question?

Mr. A. N. Talbot.—Mr. President, does the passage of the resolution carry with it the approval of this body applicable to the geographical distribution? If so, I hope it will not be carried. It is one of the best things connected with the government of the Society. It has carried weight with the members away from the City of New York more than anything connected with the Society. It has been stated here that other National Societies do not have it. My information is that they wish they had, and it has been more or less unsatisfactory because of their lack of such distribution. I think the motion should not be carried.

R. S. Buck, M. Am. Soc. C. E.—Mr. President, I ask for information; do I understand it is open to the Board to name its own lines of division, on this resolution of Mr. Whinery, or does it mean to wipe out the whole provision?

Mr. S. Whinery.—My intention was to wipe out the whole system of geographical distribution in the control of the Society, and put it precisely on the same basis as the other National Engineering Societies in that respect.

The President.—You have heard the motion, gentlemen. As many as are in favor of it signify by saying "aye"; contrary, "no". The noes seem to have it; the noes have it.

Mr. G. W. Tillson.—I move that the whole matter be referred back to the Board of Direction, with a request that it prepare what it considers a suitable amendment in view of the discussion we have had here to-day, in time to be acted upon by the next Convention.

(Motion duly seconded.)

Mr. R. L. Humphrey.—Mr. President, I submit that that is already covered by the preceding motion. It instructs the Board to do that.

A Member.—I would like to know where we are at.

A Member.—We have heard that the first motion was passed relating to the recommendations of the Board; is it a fact, or is it not, that that motion was passed, or was it made as a motion and then disposed of, or was it made as a motion and passed?

The President.—It was made as a motion, passed, and then discussed; and it was made as a motion again, and now it is being discussed again.

A Member.—The motion of the gentleman here was not the same as the gentleman made, to adopt the recommendations of the Board.

Mr. G. W. Tillson.—Mr. President, my first motion was that the report of the recommendations of the Board be approved. That I understand was carried. Now my second motion is that the matter be referred back to the Board of Direction for it to draw up what

it considers a proper amendment to carry out the general recommendations after having heard the discussion here to-day. That I do not think has been covered by any other motion.

The President.—It is the purpose of the Board in sending out this suggested amendment to accompany it with the remarks that have been made here to-day. Now, I will put this motion, in order to be sure to have all the motions on the same subject. As many as favor the motion just stated signify by saying "aye"; contrary, "no". The ayes have it.

Mr. S. Whinery.—This only postpones action in this matter, if it is to go back to the Board; if you must approach the Board through five members who petition for a certain amendment, the Board considers it and must present that to the next Annual Convention or the next Annual Meeting. It simply delays action, as I understand it, for six months.

The Secretary.—Your statement, if you will pardon me, is not a correct one. When five members suggest an amendment the Board has nothing whatever to do with it. It must go to the Society in a certain way.

Mr. S. Whinery.—Still, Mr. Secretary, does not my contention stand true that to refer this again to the Board simply means a postponement of consideration of the question for six months more?

The Secretary.—I do not think so, sir. Under the Constitution, no amendment can be voted upon and become operative until next October; that is the best that can be done.

The President.—The next thing in order is a letter from E. W. Clarke, M. Am. Soc. C. E.

The Secretary.—The letter is as follows:

"Pleasantville, N. Y.,
"December 31, 1912.

Discussion on Presentation of Papers.

"Board of Direction,
"American Society of Civil Engineers.

"Gentlemen.—It has appeared to me from a rather infrequent attendance at the semi-monthly meetings of the Society that the practice of reading papers and the discussion thereon at these meetings is of really very little value and that at this time it might be proper to suggest that such readings be discontinued.

"Those who are interested in any particular paper read it and the discussions at a time when they can get the greatest benefit from it, and more than 75% of the membership are entirely dependent on this method of using the papers. The great listlessness which is generally shown during the reading of the papers and the reluctance of the members to enter into any verbal discussion seems to me to indicate that very little interest is taken in the actual presentation at the

Discussion on Presentation of Papers (continued).

Society meetings. This is emphasized by the great rarity with which papers are presented by their authors, and it must be a great burden to the Secretary to read papers which he evidently does not have opportunity to read over beforehand.

"Personally, I generally go to these meetings in hope of seeing friends, whom I would not meet otherwise, and it seems to me that if, for instance, some informal presentation could be arranged of actual construction work illustrated by lantern slides on some big or little job by men actually engaged on the work, or some similar short entertainment, and the bulk of the evening devoted to a social meeting, it would tend to get larger attendance than is now the case.

"I request that this letter be read at the Annual Meeting, and in order to get an expression of opinion from the members that the following be offered:

"That the President appoint a committee of five to investigate and report at the regular business meeting in March, 1913, on the possibility of securing suitable engineering entertainments to take the place of the reading of the formal papers for the semi-monthly meetings.

"Yours truly,

"E. W. Clarke, M. Am. Soc. C. E."

A Member.—I second the motion.

The President.—You have heard the motion, gentlemen. Are you ready for the question?

E. W. Clarke, M. Am. Soc. C. E.—I do not wish, Mr. President, to appear in the attitude of being a critic of the reading of the papers at the semi-monthly meetings. I think they are one of the most important features of the Society; but, as I said in the letter, the Society meetings do not seem to be productive of any great interest, and while that may not be the best method of making the members take an active interest in them, it seems to me possible that some scheme might be gotten up that would get more members here, and have them display greater interest in the semi-monthly meetings, and I wrote the letter in the hope that there would be a discussion, and that some action might be taken along those lines.

O. E. Hovey, M. Am. Soc. C. E.—Mr. President, I would like very much to discuss this matter, and there are one or two things that it appears to me that I might say a word or two upon. It was my privilege to be located temporarily in the City of New York when we were located at 127 West 23d Street. I did not miss a meeting during the time I was in the city. You could not have kept me away from the meetings, unless I was sick, or something of paramount importance had occurred. They were attended generally. The papers were discussed and very well discussed, frequently with slight passages of humor,

which did not necessarily appear in the published *Proceedings* afterward. The papers were thoroughly appreciated, and discussed in a way that I and a great many of my friends enjoyed to a very great extent, and in attempting to attend the meetings recently I noticed that, while the Society has grown greater, the interest seemed to be less.

There is another point that I would like to refer to. A young man whom I have been acquainted with for several years, who is a very keen, brainy fellow, deeply interested in his work, and joined the Society with a great deal of enthusiasm, has actually resigned during the last year, because he came up here, and the meetings did not interest him, and he did not see anything to come for. He told me, not three days ago, that if anything on earth could be done to bring about interest in these meetings, he would apply for re-admittance immediately.

W. J. Boucher, Assoc. M. Am. Soc. C. E.—I agree with Mr. Clarke that the meetings are fairly attended, and there is a spirit of listlessness in the proceedings. Naturally, the experience and interest which the author would have if he were here to present it, is not present when he is not here. I have noticed lately that, for the most part, the authors, unfortunately, have been from beyond the Mississippi River, which makes it more or less difficult for them to attend; but the papers were of great value and great interest.

I am not in favor of abolishing the reading of the papers. What we want is more discussion, but, unfortunately, it would appear that the membership is afraid to discuss the papers or afraid to criticise them in any way.

In one paper, recently, the author asked that there be a free and liberal discussion of his subject, which was not very often treated in the engineering press. I told a little of what I had seen and heard, which was not much, but I was the only one of probably a hundred who rose to discuss the question. We want more discussion.

In a certain club in New York City, devoted to railroad interests particularly, I know that preparation is made in advance by calling upon well-known men who can discuss the subject, and members are asked to come prepared to discuss it. Now, our Society is big enough, and the membership in this neighborhood is large enough, that it seems to me that those qualified to speak, or those connected with that class of work, might be called upon to come prepared to add something to enhance the interest of the papers. We are not a social organization, but an engineering organization, for the dissemination of engineering knowledge.

T. C. Atwood, M. Am. Soc. C. E.—Mr. President, I wish to speak in favor of Mr. Clarke's motion. I have attended the meetings quite regularly here for the last five or six years, and it has been increasingly borne in upon me that we do not get out of the meetings

Discussion on Presentation of Papers (continued).

what we should. It is no uncommon thing for the paper on the programme to be read by title, at the suggestion of the Secretary, and for there to be practically no discussion, with the result that the meeting is all over in a few minutes.*

This gives plenty of time for social intercourse afterward, if there are sufficient members present to make it enjoyable, but, if the programme of the meeting amounts to so little, the attendance is likely to be slim also. It is true that many of the papers are long and difficult to present in an interesting manner, and that we are supposed to read the papers in the *Proceedings* and to prepare discussions beforehand, but there are a great many men who do not find time to read the papers beforehand, however good their intentions may be.

I have devoted some thought to this question, and it seems to me that we might pursue an intermediate course which should give more satisfactory results; that is, present an abstract of the paper so as to have something actually before the Society to discuss. As it is now, if the paper is not read, there is nothing directly before the meeting, and those who have not made preparation beforehand cannot be expected to discuss the paper, but if the paper is presented to them in a brief and interesting form, many could add some points from their experience which would be illuminating and make the meetings of greater interest.

The abstract of the paper should be prepared by its author, and every inducement should be made to have the authors read their own papers. If, for any reason, however, the author fails to send an abstract of his paper, such an abstract should be prepared by the Secretary and presented. Abstracts of long written discussions should also be prepared and read. Whenever possible, lantern slides should be prepared to add interest to the presentation of the paper, and the author should be allowed to have these slides prepared at the expense of the Society, the only requirement made of the author being that he furnish suitable photographs or drawings.

We all realize this morning that the acoustic properties of this hall are very poor. Hardly anything that has been said was audible over more than two-thirds of the hall. This makes it especially important that in meetings, where the interest is sure to flag when the speaker cannot be heard, the papers be read by some one who not only understands the subject and is familiar with the paper, having read it a sufficient number of times beforehand to present it to good effect, but is also endowed with a good voice and can make himself

* Mr. Atwood's attention was called to the fact that his original statement to the Annual Meeting was not in accord with the records, and an opportunity was given him to change it. While, however, he has made considerable modification in that statement, it is still in a form which, unless corrected, would give a false impression to members who have not attended meetings during the past five or six years. A statement of the facts as to the presentation of papers and discussions during that period has therefore been prepared for the information of the membership and will be found on page 103. (Secretary.)

heard all over the hall. The Society is in a prosperous condition, and we can easily afford to hire some good speaker, who is familiar with engineering work, to present the papers or their abstracts, together with the written discussions, so that every one present can hear and enjoy them.

I thoroughly believe that a more satisfactory presentation of the papers, both as to quantity and method of presentation, will make the meetings sufficiently interesting to insure a larger attendance and make both the programme and the social parts of the evening worthy of every man's attention.

F. R. Harris, M. Am. Soc. C. E.—Mr. President, I ask to have Mr. Clarke's motion read, please.

The Secretary.—Mr. Clarke's motion was that the President appoint a committee of five to investigate and report at the regular business meeting in March, 1913, on the possibility of securing suitable engineering entertainments to take the place of the reading of the formal papers for the semi-monthly meetings.

Mr. F. R. Harris.—Mr. President, I think that one of the most important functions of a technical society of this sort is the technical papers, the papers on engineering, the papers that are submitted, whether they are printed or read. There are probably two kinds of papers that you receive, one type of paper is one of general interest, one that reads well and one that brings a large attendance at the meetings. The other type of paper is probably more technical, more special, and dryer; but it is very often the case that the second type of paper is the more useful one to the members of the Profession, perhaps not at that particular time, but later, when they may want to refer to it and see what has been done on the subject. For instance, I noticed in the past year there was a paper on physical valuation. Now, I was not here when that paper was read, and it must have been extremely dry reading to most of those in attendance, but it was an extremely useful and interesting type of paper. I mention it because it occurred to me that if a committee—some auditing committee or some committee who would look over these papers—could divide the papers into two types, one paper for publication and the other paper that they thought would be of more interest and would bring a larger attendance to have it read.

Another thing is to arrange discussion by invitation in advance, and that suggests the securing of special papers by special invitation.

In connection with that there are some societies, technical societies, that offer inducements in the way of presentation of papers; that is, they pay for them, and one of the gentlemen here to-day said that the Society was so well off, and had so much of a surplus, that it might be a very good way to use some of that surplus. It would probably be impossible to pay enough to warrant a man of experience spending any great amount of time on it. I mean, you could not pay for his

Discussion on Presentation of Papers (continued).

time, but you could at least partly reimburse him for the expenses he would be put to in the preparation of the paper.

I had the experience not very long ago, in another Society, where I did not know they paid for papers. After my paper was published, I received a check for $40. It went quite a way toward paying the expense I had been put to, and was very welcome, and I think that would to some extent induce more of the members to go to the expense. I would therefore offer an amendment. Is this a resolution or a motion?

The Secretary.—A motion.

Mr. F. R. Harris.—I offer an amendment to it: Instead of the motion as it is written, that the entire subject be referred to a committee for careful investigation for recommendation and report. It seems to me that it is entirely too large and broad a question to be handled in this offhand manner; and I think the proposition—I may not be right, but it seems to suggest that we do away entirely with papers and have engineering entertainments. Engineering entertainments is a very broad term. I do not suppose that we are to have a cabaret show or anything of that sort here. It might change the entire character of this Society from a highly important professional society to an amusement society, and I do not believe any of us want that. So I would offer that as an amendment, that a committee be appointed by the President to investigate the entire subject and report and recommend what steps—by amendment to the resolution now before this Society—what steps should be taken in order to secure more interesting papers, and to take up the question also of what papers should be read, what papers should be merely published and not read, and also the question of some compensation for the publication of papers.

Mr. G. S. Williams.—Mr. President, I am to some degree in harmony with the sentiments of the last speaker, but it does not seem to me that we need to increase the interest of our papers. I think our papers are good enough. It is rather the manner of the presentation that is at fault. I certainly object to anything like the reflection that the papers that are presented before this Society are not interesting. They are interesting, the most interesting engineering reading that you can find; and it is simply a matter of presentation. I would move an amendment to substitute that a committee of five be appointed to consider an improvement in the methods of the presentation of papers before this Society.

(Motion duly seconded.)

A Member.—I move an amendment that the whole matter be referred to the Board of Direction.

Mr. W. J. Boucher.—Mr. President, the Board of Direction probably has enough to do without this. A committee of five can handle it.

The President.—Will you state that motion again, Mr. Williams?

Mr. G. S. Williams.—That a committee of five be appointed to consider and report upon an improvement of the methods of the presentation of papers before this Society.

The President.—That you move as a substitute motion?

Mr. G. S. Williams.—I move that as a substitute for the original motion.

The President.—Gentlemen, you have the motion. As many as are in favor of it signify by saying "aye"; contrary, "no". It is so ordered.

The next thing in order is a letter from Percival M. Churchill, Assoc. M. Am. Soc. C. E., asking that action be taken to appoint a committee to draw up a form for establishing an exchange for the marketing of engineering services of every sort.

Letter Relating to Marketing of Engineering Services.

The Secretary.—Mr. Churchill's letter is as follows:

"Elmwood, Mass., January 7, 1913.

"*The Secretary,*

"*American Society of Civil Engineers,*

"*220 West Fifty-seventh Street, New York City.*

"Dear Sir.—As I do not expect to be present, will you kindly present for me at the Annual Meeting the following:

"*Moved:* That the Board of Direction be instructed to confer with the officers of the Am. Institute of Consulting Engineers, the Am. Soc. of Mechanical Engineers, the Am. Institute of Electrical Engineers, and of any other Engineering Society it deems proper to include, for the purpose of ascertaining if these Societies will join with us in appointing a joint Committee to draw up a plan for the establishment and operation of an Exchange for the Marketing of Engineering Services of every sort. The Board of Direction to be authorized and instructed to appoint the Committee from this Society as soon as any of the other Societies signify their willingness to join in the work.

"This motion is now offered because the time has arrived when engineers as a body can no longer delay with dignity and in justice to the profession and to the public the performance of this important duty.

"What is here proposed will develop far beyond the scope of an employment bureau. It is the handling in a scientific and business-like manner the problem of the proper supervision of the engineering work of the country. It is applying "Scientific Management" to the expenditure of the wealth of the public by selecting competent men to direct that expenditure.

"Our present lack of system is extremely wasteful of the engineering talent of the country, which costs so much to train, and wasteful of vast amounts of public wealth thro its expenditure being placed in incompetent hands. For example, outside of the cities and the State work the greater part of the money spent in this country on highways is wasted thro incompetent handling, while at the same time hundreds of engineers who could save most of this waste are idle, because there is no work for them. Cities and towns are wasting large

Letter Relating to Marketing of Engineering Services (continued).

amounts in construction, and later in avoidable repairs on waterworks, sewers, lighting plants, etc., because they do not know how to select proper engineering supervision, or do not even realize that engineering supervision is necessary. Enormous wealth has been wasted thro needless litigation over water rights in the West, which could have been almost entirely avoided had the engineers of this country taken prompt, persistent, combined action to bring about a proper settlement thro the education of the public. And engineers as a body are responsible if this waste is allowed to continue, for they have the knowledge to work out and the power to put in operation the remedy.

"The proposed motion involves one of the most important steps in applying the needed remedy. Not only should this bureau properly distribute the available engineering talent to the available work, but it should conduct an active progressive campaign of education among State, City and Town officials, among business firms and among the public, to demonstrate what they would gain by placing their work in the hands of competent engineers. It would teach these parties that it would provide the proper talent for any kind of engineering work, and also what is a fair and proper compensation for this service. It should and could raise the public estimate of the engineer to a point where the public engineering work would be trusted to him rather than to lawyers, merchants, and barbers, as is the case at present. It would undoubtedly to a considerable extent check the present threatened absorption of the cream of the engineering work by a few large engineering corporations.

"The Committee of this Society which was appointed to investigate engineering conditions will furnish valuable data with which to work out the details of this plan. The need for its establishment is here, and this motion is the first step toward its accomplishment. Let us take it now.

"Respectfully,

"PERCIVAL M. CHURCHILL."

A MEMBER.—I move that the matter be referred to the Board of Direction.

(Motion duly seconded.)

THE PRESIDENT.—You have heard the motion, gentlemen. As many as favor that signify by saying "aye"; contrary, "no". It is so ordered.

Letter Relating to Proposed Code of Ethics.

THE SECRETARY.—I have still another letter from Mr. Churchill dated a day later.

"ELMWOOD, MASS., JANUARY 8, 1913.

"The Secretary,
"American Society of Civil Engineers,
"220 West Fifty-seventh Street, New York City.

"DEAR SIR.—Several of the large engineering societies have recently adopted Codes of Ethics. The Am. Soc. of M. E. has a Code under consideration now which is the result of a study of those of the other Societies and which was drawn up with the idea of its being applicable to engineers generally. Two out of the five members of the Committee which had this matter in charge are also Members of this Society.

"Under these circumstances it would seem proper to consider the adoption of this Code as it stands without the formality of submitting it to a Committee of this Society. I therefore wish to make the following motion:

"*Moved:* That this Society shall consider the adoption of the Code of Ethics for Engineers recently proposed by a Committee of the American Society of Mechanical Engineers and published in *Engineering News,* January 2, 1913.

"That the proposed Code be printed as a letter ballot and submitted to the members of this Society with the request that each member vote to accept or reject the Code article by article; that where a member so desires he shall—after voting against a certain article—submit a substitute in the form he desires that article to take. In the same manner additions may be presented.

"This ballot to be closed on April 1st, 1913.

"The Board of Direction shall then send out a second letter ballot giving the Code as first proposed and also any suggested changes and additions. Members shall again vote article by article. The ballots to be opened at the next Annual Convention and any article having a majority of the votes cast shall be declared adopted.

"The Code as thus adopted to be then printed in the *Proceedings* and the *Transactions* of the Society.

"Respectfully submitted,

"PERCIVAL M. CHURCHILL, ASSOC. MEMBER."

THE PRESIDENT.—What is your pleasure?

MR. G. S. WILLIAMS.—I move that the motion be referred to the Board of Direction.

(Motion duly seconded.)

THE PRESIDENT.—Gentlemen, you have heard the motion. All in favor of it signify by saying "aye"; contrary, "no". It is so ordered.

The next thing in order is the matter of the International Engineering Congress. Proposed International Engineering Congress.

INTERNATIONAL ENGINEERING CONGRESS—1915.

For the information of the membership of the Society, the Secretary begs leave to report as follows:

That early in 1912 engineers belonging to this and other National organizations resident on the Pacific Coast, suggested the holding of a National Engineering Congress in San Francisco in 1915, during, and in connection with, the Panama-Pacific Exposition. A plan for the organization of this Congress which provided that each of the Societies should separately undertake the work of the Congress in its special line of engineering was presented, and this Society was asked to undertake the carrying out of that part of the Congress relating to Civil Engineering.

At the request of the Board of Direction, the Secretary visited San Francisco in February and March, 1912, and conferred with mem-

Proposed International Engineering Congress (continued).

bers of this Society, and of the other Societies there, with the result that the plan of organization was entirely changed to the following:

The whole scheme was to be underwritten as follows:

To the amount of $10 000 at least, by subscription on the Pacific Coast, and each of the five National Societies to underwrite the scheme for the following amounts: American Society of Civil Engineers, $9 000; American Institute of Electrical Engineers, $9 000; American Society of Mechanical Engineers, $5 000; American Institute of Mining Engineers, $5 000; and the Society of Naval Architects and Marine Engineers, $2 000.

This would make up a total Guarantee Fund of $40 000, which it was believed would be more than ample to defray any extra expenses.

All of the Societies above mentioned have, at the present time, agreed to underwrite the amount specified, except the American Institute of Electrical Engineers. That Society, having already agreed to conduct an Electrical Congress, did not feel that it could do quite so much for the project, but readily agreed to furnish $3 500.

The $10 000 from the Pacific Coast has already been subscribed, and we are informed that this Fund can probably be increased without difficulty to $5 000 more, so that there is no question that the financial backing of the proposed Congress is assured.

The general idea of the Congress is that its organization and entire management shall be in the hands of a General Committee, in which each of the five Societies undertaking the Congress shall have an equal representation, namely, six members each (except the American Institute of Electrical Engineers, which will have four), making a total Committee of twenty-eight members; that two of the members representing each Society shall be, *ex-officio*, its President and Secretary, and that the other four members shall be selected from the active membership of each Society resident in or near San Francisco.

The resulting publications are to be issued as a whole, and forwarded to each engineer who becomes a member of the Congress; and all American, as well as Foreign Engineers, are to be invited to join the Congress upon payment of a small sum sufficient to cover the cost of the issue to them of all of the papers and discussions received.

It appears at the present time that the holding of this Congress is assured.

The President.—Any new business?

Lunch at Annual Meeting.

Mr. H. M. Wilson.—Mr. President, I have a matter of old business that I would like to bring up at this festive hour of noon, and I do it with some hesitancy. I was asked by several men, immediately upon my entrance to the building this morning, as to what action had been taken by the Board in the matter of our arranging to lunch somewhere; and I ask as a matter of information, because I have not

heard of any action of the Board. What action was taken in connection with the recommendation last year that the Board consider the matter of arranging for luncheon?

THE SECRETARY.—Mr. President, the resolution adopted at the last meeting was considered by the Board of Direction, and the Board decided—I have not the Board's minutes here, unfortunately, or I would give the exact wording—that it was not advisable to undertake the serving of a lunch here in this building at the time of this meeting.

MR. H. M. WILSON.—Was the reason assigned the inadequacy of the accommodations here?

THE SECRETARY.—That is what it is.

MR. H. M. WILSON.—Mr. President, I move that the Board of Direction be, and hereby is, authorized to consider plans for increasing the capacity of the Society House, so that it may accommodate the members at annual and other meetings.

THE PRESIDENT.—Is there a seconder to that motion?

MR. T. C. ATWOOD.—I second the motion. It stands now ready for discussion. I move to amend that motion, Mr. President, by requesting the Board of Direction to secure suitable accommodations for the Annual Meeting, either by increasing the accommodations in this building or securing suitable temporary accommodations elsewhere. It does not seem that it will be absolutely necessary perhaps to increase this building at large cost, simply for one meeting in the year.

THE PRESIDENT.—Mr. Wilson, will you please state your motion?

MR. H. M. WILSON.—That the Board of Direction be, and is hereby, authorized to consider plans—that does not necessarily mean building plans—for increasing the capacity of the Society House to better accommodate the membership at Annual Meetings and other meetings; and I think such an amendment, or any other plan that would meet this purpose, which would better accommodate the members at the Annual Meetings would be wise.

THE PRESIDENT.—There does not seem to be any objection to having the Board of Direction consider that matter. As many as favor that motion signify by saying "aye"; contrary, "no". The ayes seem to have it; the ayes have it.

Acoustics of Auditorium.

H. H. QUIMBY, M. AM. SOC. C. E.—Mr. President, I would like to bring up an old matter, which has never been satisfactorily disposed of. I think it is a reproach to a Society so scientific as ours is, and which has such a surplus of revenue as ours seems to have, that it continue to maintain an auditorium with such uncommonly poor acoustic properties as this room has.

I remember the very first meeting we had in this room; it was not as large as this. It was very soon perceived that members were changing their seats to hear what was being said by the different members, and attention was called to the matter, and somebody asked whether

Acoustics of Auditorium (continued).

something could not be done to improve these acoustic defects. Some member suggested that wire might be hung across the room, and some man misunderstood the meaning of the suggestion and approved it on the theory that it was a motion to hang the architect.

To-day I noticed that a speaker in the rear of the room was speaking while the President was, and apparently neither realized that the other was speaking. Clearly, Mr. Clarke did not realize that the President was speaking at the same time.

Now, I move that the Board of Direction be requested by this meeting to employ the best talent that is available and do something to make it more satisfactory to the hearers in this room and to the speakers also.

(Motion duly seconded.)

THE PRESIDENT.—Gentlemen, you have heard the motion that the Board of Direction be authorized to employ an expert to consider the matter of improving the acoustics of this hall. As many as are in favor of that signify by saying "aye"; contrary, "no". It is carried.

Ballot for Officers.

The next in order is the announcement of the result of the ballot, the report of the tellers to canvass ballots for officers.

THE SECRETARY.—The tellers report as follows:

For President:

	GEORGE F. SWAIN	1 724
	Scattering	11
For Vice-Presidents:		
	J. WALDO SMITH	1 713
	CHARLES H. RUST	1 716
	Scattering	12
For Treasurer:		
	JOHN F. WALLACE	1 724
	Scattering	9
For Directors:		
District No. 1.	HENRY W. HODGE	1 674
	JAMES H. EDWARDS	1 666
	Scattering	7
District No. 2.	LEONARD METCALF	1 669
	Scattering	8
District No. 4.	HENRY R. LEONARD	1 662
	Scattering	10
District No. 5.	EDWARD H. CONNOR	1 668
	Scattering	9
District No. 7.	SAMUEL H. HEDGES	1 668
	Scattering	8

THE PRESIDENT.—The report read by your Secretary shows that by your ballot you have elected George F. Swain as President, J. Waldo Smith and Charles H. Rust as Vice-Presidents, and John F. Wallace as Treasurer. Officers Elected.

Directors for District No. 1, Henry W. Hodge and James H. Edwards.

District No. 2, Leonard Metcalf.

District No. 4, Henry R. Leonard.

District No. 5, Edward H. Connor.

District No. 7, Samuel H. Hedges.

I would ask Mr. Stearns and Mr. Noble to escort the President-elect to the platform.

PRESIDENT GEORGE F. SWAIN.—Fellow members of the American Society of Civil Engineers: I have no words to express adequately my appreciation of the great honor which you have done me in electing me to this office. I esteem it as the greatest honor that could come to an engineer in this country. As I recall the long roll of distinguished men who have occupied the position in the past, and as I look about me and see how many more there are equally distinguished, who are eminently worthy of it, I am at a loss to understand really how it has happened, and it is beyond my powers of analysis to figure it out. Remarks by President Swain.

Of one thing, however, I feel very sure, and that is that there is no member of this Society who has a higher ideal than I of the work of the engineer or of its value and consequence. I believe that the engineer, more than any other man, is the real civilizer of the world. It has always seemed to me that advances in civilization depended upon progress in material ways rather than upon advances in moral ideas.

The moral principles which should guide a man in the world were formulated certainly nineteen centuries ago, and may be found expressed in the writings of the old moralists in as complete a form as in any modern treatise; but in those days, of course, those principles were accepted and adhered to by few; and cities were arrayed against cities in warfare, tribe against tribe, and clan against clan; but the man who perfects a new system of transportation, or a new motive power, or a new means of abridging space and time, the man who harnesses the waterfall or the power of the sun's heat, which has been stored in the past, and makes those forces available in the manufacture of articles which conduce to the comfort of mankind, and which are needed in the distant countries to which they can be easily transported, or the man who by improving sanitary conditions conserves the life of the community—such a man, it seems to me, is doing a work which is really making the whole world akin, and conducing to a broader sympathy and the universal brotherhood of man; and therefore laying the foundations for advance in civilization and for true moral progress.

President's Remarks (continued).

Holding this in view, you can perhaps understand how deeply I esteem the honor of being President of what I hold to be the chief engineering society of this country, certainly the oldest.

I also thank you gentlemen because this act of yours is probably largely, if not mostly, due to a recognition of a branch of the Profession which has not hitherto been recognized in this way at your hands—a branch to which a large part, although not the largest part, of my life has been devoted, and that is the Profession of the engineering teacher. If we believe in the high mission of the engineer, we must necessarily believe that he must have the highest and best training which is possible—not that such training and education are always most evident in the schools—far be it from us to maintain that attitude. We have too many shining examples to the contrary to hold it; but, however or wherever such education is obtained, we must, as a Society, hold that the engineer should endeavor to get the highest and best that there is.

I thank you, therefore, for your recognition of this fact. In assuming office, gentlemen, I do so, not only with a high sense of the honor which you have conferred upon me, but also with a high sense of the responsibilities. Promises are easily made and easily broken, and all I can say is that as far as in me lies, and to the best of my ability, I will aim to promote the highest ideal of the Profession and to maintain the high standard which this Society has always maintained. I thank you, gentlemen.

The only duty which remains for me to perform at this time, I am informed, is to state that there will be a meeting of the Board of Direction, downstairs in the Secretary's office, immediately upon the adjournment of this meeting.

A MEMBER.—I move to adjourn.

(Motion duly seconded.)

Adjourned.

THE PRESIDENT.—It is moved and seconded that we adjourn. All in favor say "aye"; contrary, "no". It is a vote.

Data Relating to the Presentation of Papers and Discussions at Ordinary Meetings of the Society for the Six Years, 1907 to 1912, Inclusive.*

During 1907 : 12 Papers were presented by Author (or his substitute),
8 " were read by Secretary,

20 Papers, none of which was presented "by title,"
40 Written discussions were read by Secretary,
10 " " " not read, first on account of lack of time; second, too mathematical to read,
60 Members discussed orally.

There was no meeting during 1907 at which a paper was not read—and no meeting at which either oral or written discussion was not presented.

During 1908 : 11 Papers were presented by Author,
6 " " read by Secretary,
1 Topical Discussion opened by a Member,
1 Address by a Member,

19 Papers, none of which was presented "by title,"
15 Written discussions read by Secretary,
7 " " presented "by title," on account of lack of time,
74 Oral discussions.

There was no meeting during 1908 at which a paper was not read—and no meeting at which either oral or written discussion was not presented.

During 1909 : 18 Papers presented by Author (or his substitute),
7 " read by Secretary,
4 " presented "by title,"

29 Papers, only 4 of which were presented "by title,"
36 Written discussions read by Secretary,
69 Oral discussions.

There was no meeting during 1909 at which a paper was not read—and no meeting at which either oral or written discussion was not presented.

During 1910 : 15 Papers presented by Author (or his substitute),
8 " read by Secretary,
5 " presented "by title,"

28 Papers, only 5 of which were presented "by title,"
23 Written discussions read by Secretary,
16 " " presented "by title," all on account of lack of time.
54 Oral discussions.

There was no meeting during 1910 at which a paper was not read—and no meeting at which either oral or written discussion was not presented.

During 1911 : 9 Papers were presented by Author (or his substitute),
12 " read by Secretary,
4 " presented "by title,"

25 Papers, only 4 of which were presented "by title,"
55 Written discussions read by Secretary,
5 " " presented "by title," all on account of lack of time,
57 Oral discussions.

There was no meeting during 1911 at which a paper was not read—and no meeting at which either oral or written discussion was not presented.

During 1912 : 9 Papers presented by Author,
9 " read by Secretary,
7 " presented "by title" only,

25 Papers, 7 of which were presented "by title,"
40 Written discussions read by Secretary,
2 " " presented "by title," on account of lack of time,
82 Oral discussions.

During 1912 there were two meetings at which papers were not read, but there was good reason for this. At one of these the paper was a very long one, and there was some written discussion and considerable oral discussion. At the other meeting, there were two papers, one of which contained a great deal of tabular matter, and there were four written discussions read by the Secretary, as well as a great deal of oral discussion.

It will be seen from the above that during the six years covered by this statement, 146 papers were presented ; 74 of these were presented by the author or by his representative; 50 were read in whole or in part by the Secretary, one introduced a topical discussion and one was a special address by a member. There were only 20 papers presented "by title."

Lantern slides were used to illustrate 53 of the 146 papers mentioned above.

During the six years there were only two meetings at which papers were not read, and the reason for this is clearly shown above.

* See reference on page 92.

EXCURSIONS AND ENTERTAINMENTS AT THE SIXTIETH ANNUAL MEETING.

Wednesday, January 15th, 1913.—In the afternoon, through the kindness of George W. Kittredge, M. Am. Soc. C. E., Chief Engineer of the New York Central and Hudson River Railroad, and G. A. Harwood, Chief Engineer of the Electric Zone Improvements, a large party was enabled to inspect the Grand Central Terminal Improvements. About 300 members and guests assembled at 2.30 P. M. at the temporary concourse of the Grand Central Palace, where they were met by representatives of the Engineering Department, under whose guidance many small parties inspected the Suburban Level, the Trucking Subways, Pipe Galleries, Service Plant for Light and Heat, Shops, Station Building, and many of the minor details of this extensive improvement.

At 9 P. M. there was a Reception, with dancing, in the Society House, at which there were present 400 members and about 300 ladies and other guests. Supper was served during the evening.

Thursday, January 16th, 1913.—By invitation of the Contractor, H. S. Kerbaugh, Inc., the members of the Society and their guests made an excursion to the Kensico Dam, one of the important features of the new Catskill water supply for New York City. The party (of about 600) was conveyed to Valhalla, near the site of the dam, in a special train which left the Grand Central Station at 10.15 A. M.

The dam crosses the valley of the Bronx River, about three miles north of White Plains, and will form a storage and distributing reservoir for the water impounded in the Catskills. The total capacity of the reservoir will be about 38 billion gallons, equivalent to a continuous supply for New York City for about 50 days at 600 million gallons daily. The maximum depth of water behind the dam will be 155 ft., and at the normal flow line elevation, 355 ft. above mean sea level, the reservoir will cover 2 218 acres. The total area acquired by the city—4 5[illegible] acres—provides a marginal protective strip averaging 500 ft. wide around the entire flow line.

The Kensico Dam will be a gravity structure of cyclopean masonry, 1 843 ft. long, with a maximum height of about 300 ft. It will be more than 170 ft. high for 1 000 ft. of its length. Its top width under the coping will be 28 ft., and the width of the base of the maximum section will be more than 250 ft. It will contain about 1 000 000 cu. yd. of masonry. The up-stream face will be of concrete blocks, and the down-stream face of granite. The entire dam will be divided into sections by transverse expansion joints about 79 ft. apart longitudinally.

After inspecting the excavation for the dam (about 50% completed) and part of the Contractor's plant, the party was entertained at

luncheon as the guests of the Contractor. After luncheon a visit was made to the quarry and the stone-crushing plant.

The party left Valhalla on the special train at 4.30 P. M. and arrived at the Grand Central Station at 5.30 P. M.

In the evening, at the Society House, there was an informal "Smoker" at which 800 members and guests were present.

The following list contains the names of 815 members who registered during the Annual Meeting. The list is not complete, however, as many members failed to register, and it does not contain the names of any of the guests of the Society or of individual members. It is estimated that there was an attendance of 400 ladies and other guests, making the total attendance at the Annual Meeting more than 1 200.

Abbott, C. P. . . White Plains, N. Y.
Adams, E. G., Jr. . . Waverly, N. Y.
Affelder, L. J . . . . . Pittsburgh, Pa.
Aiken, W. A. . . . . Philadelphia, Pa.
Aims, W. I . . . . New York City
Akerly, H. E . . . . . Rochester, N. Y.
Alexander, H. J., White Plains, N. Y.
Allaire, D. A . . . . . . Brooklyn, N. Y.
Allen, C. Frank . . . . . Boston, Mass.
Allen, C. M . . . . . . Worcester, Mass.
Allen, C. R., Jr., Saratoga Springs, N. Y.
Allen, E. Y. . South Orange, N. J.
Allen, Kenneth . . . New York City
Allen, Walter Hinds, Brooklyn, N. Y.
Ammann, O. H. . Philadelphia, Pa.
Appleton, T. A . . . . . Beverly, Mass.
Archer, A. R. . . . Philadelphia, Pa.
Armstrong, G. S., Jr., Whitinsville, Mass.
Armstrong, R. W. . New York City
Arnold, W. H . . . . . . New York City
Ashbaugh, L. E . . . . New York City
Atwood, T. C . . . . . Yonkers, N. Y.
Auryansen, F . . . . . Jamaica, N. Y.

Babcock, W. S . . . . . New York City
Baird, H. C . . . . New York City
Baker, Ira O . . . . Urbana, Ill.
Baldwin, F. H . . . . . Bayonne, N. J.
Baldwin, T. A . . . . . . Boston, Mass.
Baldwin, W. J . . . . Brooklyn, N. Y.
Bamford, W. B . . . . . Belmar, N. J.
Banks, C. W. . Pleasantville, N. Y.
Barker, C. W. T. . Philadelphia, Pa.
Barker, J. M . . . . Boston, Mass.
Barnes, M. G . . . . . . Albany, N. Y.
Barnes, W. T . . . . Chicago, Ill.
Barnett, R. P . . . . . New York City
Barney, P. C . . . . . . Brooklyn, N. Y.
Barney, W. J . . . . . . New York City
Barrett, R. E . . . . Boston, Mass.
Bascome, W. R. . . . New York City
Basinger, J. G. Forest Hills, N. Y.
Bates, Onward . . . . Chicago, Ill.
Bayley, C. A. D. . . Montclair, N. J.
Beach, W. N . . . . . . New York City
Beaty, R. E . . . . New York City
Becker, E. J. . . Schenectady, N. Y.
Becker, R. C . . . . New York City
Beebe, J. C . . . . Butte, Mont.
Beekman, J. V., Jr. . Boston, Mass.
Bellows, O. F . . . . Rochester, N. Y.
Bellows, S. R . . . . . . New York City
Belzner, Theodore. . New York City
Bennett, W. B . . . . Harrisburg, Pa.
Bensel, J. A . . . . Albany, N. Y.
Bentley, J. C. . . Watervliet, N. Y.
Berger, Bernt . . . . . . New York City
Berger, John . . . . New York City
Beswick, J. E., Tompkinsville, N. Y.

Bettes, C. R.,
Far Rockaway, N. Y.
Bevan, L. J.......New York City
Blair, C. M....New Haven, Conn.
Blakeley, G. H.,
South Bethlehem, Pa.
Blakeslee, C...New Haven, Conn.
Blakeslee, H. L.,
New Haven, Conn.
Blanchard, A. H...New York City
Blatt, Max.........New York City
Boardman, H. S......Orono, Me.
Bogert, C. L.......New York City
Boller, A. P., Jr.,
East Orange, N. J.
Booth, G. W......New York City
Boright, W. P....Chatham, N. Y.
Boucher, W. J....New York City
Boughton, W. H.,
Poughkeepsie, N. Y.
Bowers, George.....Lowell, Mass.
Bowman, A. L....New York City
Bowman, D. W..Phœnixville, Pa.
Boyd, R. W.......New York City
Brace, J. H.,
Shelburne Falls, Mass.
Brackett, Dexter....Boston, Mass.
Bradley, F. E......New York City
Brainard, A. S.,
East Hartford, Conn.
Brainard, Owen....New York City
Bramwell, G. W...New York City
Breitzke, C. F.....New York City
Breuchaud, Jules...New York City
Breuchaud, J. R...New York City
Brewer, Bertram..Waltham, Mass.
Briggs, W. C.....Brooklyn, N. Y.
Brink, L. C.......New York City
Brodie, O. L.,
West New Brighton, N. Y.
Brooks, Frederick...Boston, Mass.
Brooks, J. P......Potsdam, N. Y.
Brown, B. S........Boston, Mass.
Brown, D. H........Alpine, N. J.
Brown, R. H......New York City
Brown, S. P.,
Montreal, Que., Canada
Brown, T. E......New York City
Bruning, H. D....Columbus, Ohio
Brush, W. W.....Brooklyn, N. Y.
Bryson, Andrew..New Castle, Del.
Buck, H. R......Hartford, Conn.
Buck, R. S...San Francisco, Cal.
Buettner, O. G. H..New York City
Burden, James.....Oswego, N. Y.
Burdett, F. A.....New York City
Burgess, G. H......Albany, N. Y.
Burpee, G. W.....New York City
Burpee, Moses......Houlton, Me.
Burr, W. H.......New York City
Burroughs, H. R...New York City
Bush, E. W.........Lyme, Conn.
Bush, Lincoln..East Orange, N. J.

Cadwallader, W. L.,
New Rochelle, N. Y.
Cain, William..Chapel Hill, N. C.
Cantwell, H. H....Yonkers, N. Y.
Carey, E. G.......New York City
Carle, N. A........Newark, N. J.
Carpenter, C. E...Yonkers, N. Y.
Carr, Albert..East Orange, N. J.
Carter, C. E......Reading, Mass.
Casani, A. A......New York City
Chadbourn, W. H.,
Brookline, Mass.
Chandler, E. L.,
New London, Conn.
Chappell, T. F.....New York City
Chase, C. F...New Britain, Conn.
Chase, W. H..New Bedford, Mass.
Christian, G. L...New York City
Christie, W. W....Paterson, N. J.
Church, E. C.....New York City
Churchill, J. P.,
East Orange, N. J.
Clapp, F. L......Cornwall, N. Y.
Clapp, S. K..West Shokan, N. Y.
Clark, A. E.......New York City
Clarke, G. C......New York City

Class, C. F. . . . . New York City
Clermont, J. B. . . . New York City
Coe, Robert. . . . . . . Pittsburgh, Pa.
Coffin, T. DeL. . . . Katonah, N. Y.
Cohen, F. W.,
Upper Montclair, N. J.
Cole, E. S. . . . . New York City
Cole, G. N. . . . . New York City
Cole, H. J. . . . . Montclair, N. J.
Collier, B. C. . Pleasantville, N. Y.
Coltman, Robert, Jr.,
Yonkers, N. Y.
Comber, S. X. . . . . . New York City
Connell, H. L. . . . . New York City
Connor, E. H. . Leavenworth, Kans.
Constant, F. H.,
Minneapolis, Minn.
Cook, J. H. . . . . . . Paterson, N. J.
Coombs, A. W. . . . . New York City
Coombs, S. E. . . . . . Yonkers, N. Y.
Cooper, D. R. . . . . . New York City
Cornell, J. N. H. . . New York City
Coyne, H. L. . . . . . . New York City
Crandall, C. L. . . . . Ithaca, N. Y.
Crane, A. S. . . . . . . New York City
Crane, F. E. . . . Amsterdam, N. Y.
Crehore, W. W. . . . New York City
Creuzbaur, R. W. . . New York City
Crooks, C. H. . . . . . New York City
Crosby, Hewitt. . . . . New York City
Crosby, W. W. . . . . Baltimore, Md.
Crowell, Foster. . . . New York City
Cummin, Hart. . . . . . Dayton, Ohio
Cummings, Noah. . New York City
Cummings, R. A. . . Pittsburgh, Pa.
Currier, C. G. . . . . New York City
Curtis, V. P. . . . . Worcester, Mass.

Dahl, S. T. . . . . New York City
Dakin, A. H., Jr. . . New York City
Davies, J. P. . . . . . . New York City
Davis, A. P. . . . Washington, D. C.
Davis, C. E. . . . . Philadelphia, Pa.
Davis, J. L. . . . Mt. Vernon, N. Y.
Dean, A. W. . . . . Boston, Mass.
Deans, J. S. . . . . . Phœnixville, Pa.
Dennis, W. F. . Washington, D. C.
Deyo, S. LeF. . . . . . New York City
Dilks, L. C. . . . . New York City
Dimon, D. Y. . . . . . . Passaic, N. J.
Dixon, G. G. . . . . Akron, Ohio
Donham, B. C. . Glen Ridge, N. J.
Dorrance, W. T. . . . . Boston, Mass.
Dougherty, R. E. . New York City
Dufresne, A. R.,
Ottawa, Ont., Canada
Duggan, G. H.,
Montreal, Que., Canada
Dunn, H. L. . New London, Conn.
du Pont, Biderman,
Greenville, Del.
Durham, H. W. . . . New York City

Earle, Thomas. . . . . . Steelton, Pa.
Easby, M. W. . . . Philadelphia, Pa.
Eckersley, J. O. . . . New York City
Eddy, H. P. . . . . Boston, Mass.
Edmondson, R. S. . New York City
Edwards, D. G. . . . Brooklyn, N. Y.
Edwards, J. H. . . . . . Passaic, N. J.
Edwards, W. R. . . . Baltimore, Md.
Ehrbar, L. H. . . . . . New York City
Ehrsam, Fritz. . . . . New York City
Eide, Torris. . . . . . . New York City
Ellendt, J. G. . . . Rochester, N. Y.
Elliott, C. G. . . Washington, D. C.
Ellis, H. C. . . White Plains, N. Y.
Ellis, J. W. . . . . Woonsocket, R. I.
Elwell, C. C. . . New Haven, Conn.
Ely, C. B. . . . . Harrisburg, Pa.
Ely, G. W., Jr. . . . . New York City
Endicott, M. T. . Washington, D. C.
English, H. L. . . . . New York City
Entenmann, P. M.,
Brooklyn, N. Y.
Evans, R. R. . . . . Haverhill, Mass.

Farley, M. M. . White Plains, N. Y.
Farnham, A. B. . . Pittsfield, Mass.
Fay, F. H. . . . . Boston, Mass.

Federlein, W. G...New York City
Fehr, H. R...........Easton, Pa.
Fenton, L. G.....Brooklyn, N. Y.
Ferguson, J. N.....Boston, Mass.
Fetherston, J. T., New Brighton, N. Y.
Firth, E. W......Jamaica, N. Y.
Fisher, E. A....Rochester, N. Y.
Fitch, C. L......Brooklyn, N. Y.
FitzGerald, Desmond, Brookline, Mass.
Fletcher, Robert..Hanover, N. H.
Flinn, A. D.......Yonkers, N. Y.
Follett, W. W......El Paso, Tex.
Forbes, F. B.......New York City
Ford, H. C........New York City
Ford, W. H......Philadelphia, Pa.
Foss, F. E.........New York City
Foster, E. H......New York City
Fox, W. F........New York City
Franklin, C. M....New York City
Fraser, C. E.......New York City
French, A. W...Worcester, Mass.
French, Halsey.....New York City
French, J. B......New York City
Frost, G. S.......Brooklyn, N. Y.
Fuller, A. H.......Seattle, Wash.
Fuller, G. W......New York City
Fulweiler, W. H., West Chester, Pa.
Furber, W. C....Philadelphia, Pa.

Gahagan, W. H...Brooklyn, N. Y.
Gandolfo, J. H....New York City
Gardiner, F. W....Yonkers, N. Y.
Gardner, Warren..New York City
Garfield, C. A., White Plains, N. Y.
Garrison, F. L..Philadelphia, Pa.
Gartensteig, Charles, New York City
Gaston, L. P......New York City
Gerber, Emil......Pittsburgh, Pa.
Germer, W. E.....New York City
Gifford, G. E......New York City
Giles, Robert.......New York City
Gillen, W. J.......New York City
Gillespie, R. H....New York City
Gilman, Charles..Plainfield, N. J.
Glander, J. H., Jr..New York City
Godfrey, S. C..West Point, N. Y.
Golding, T. W....Brooklyn, N. Y.
Goldsborough, J. B., New York City
Goodell, J. M., Upper Montclair, N. J.
Goodman, Joseph..New York City
Goodsell, D. B.....New York City
Gow, C. R.........Boston, Mass.
Graham, C. H.....New York City
Gray, J. L........Bayonne, N. J.
Gray, William.....New York City
Greathead, J. F...New York City
Greene, Carleton...New York City
Greene, G. S., Jr..New York City
Greenlaw, R. W...New York City
Gregg, J. H. C...Brooklyn, N. Y.
Gregory, C. E....Mt. Kisco, N. Y.
Greiner, J. E......Baltimore, Md.
Grimes, E. L.........Troy, N. Y.
Gutman, David....New York City

Haines, E. G....Brooklyn, N. Y.
Hale, H. M......Brooklyn, N. Y.
Hale, R. A.......Lawrence, Mass.
Hall, M. W.......New York City
Hall, W. M..Parkersburg, W. Va.
Hallihan, J. P....New York City
Hallock, J. C......Newark, N. J.
Halsey, W. H., Bridge Hampton, N. Y.
Hamilton, J. W...New York City
Hammel, E. F.....New York City
Hammond, J. F., Richmond Hill, N. Y.
Hanavan, W. L..Newburgh, N. Y.
Hansel, Charles...Cranford, N. J.
Harby, Isaac, Richmond Hill, N. Y.
Harding, H. S...New York City

Haring, Alexander, New York City
Harrington, F. F....Norfolk, Va.
Harris, F. R.....Brooklyn, N. Y.
Harte, C. R...New Haven, Conn.
Hartman, A. F......Nutley, N. J.
Hartwell, Harry..New York City
Harwi, S. J......Bayonne, N. J.
Hasbrouck, Oscar..Albany, N. Y.
Haskins, W. J....New York City
Hathaway, C. M., New Bedford, Mass.
Hatton, T. C....Wilmington, Del.
Hauck, William...New York City
Hayes, R. D.....Stillwater, N. Y.
Hayes, S. W......Richmond, Ind.
Hazard, Schuyler...Albion, N. Y.
Hazen, Allen......New York City
Hazen, J. V......Hanover, N. H.
Healy, J. R.......New York City
Heilbronner, L. C., Schenectady, N. Y.
Hench, N. M.....Pittsburgh, Pa.
Henderson, J. T..Hartford, Conn.
Hering, Rudolph...New York City
Hermanns, F. E...New York City
Hewes, V. H......New York City
Heyman, William, Jersey City, N. J.
Higgins, C. H..Jersey City, N. J.
Higgins, J. W., Roselle Park, N. J.
Hildreth, J. L., Jr., Cornwall, N. Y.
Hill, A. B.....New Haven, Conn.
Hill, W. R........Albany, N. Y.
Hillyer, W. R., Port Richmond, N. Y.
Hilton, J. C., Shelburne Falls, Mass.
Hilts, H. E..New Rochelle, N. Y.
Hines, H. E....Princeton, W. Va.
Hirst, Arthur.....Trenton, N. J.
Hitchcock, F. C...New York City
Hoag, S. W., Jr...New York City
Hodgdon, B. A....New York City
Hodge, H. W......New York City
Hogan, J. P....High Falls, N. Y.
Holbrook, A. R....New York City
Holbrook, Percy....New York City
Holden, C. A....Hanover, N. H.
Holland, C. M....Brooklyn, N. Y.
Holtzman, S. F....New York City
Honness, G. G...Kingston, N. Y.
Hood, J. N.....Newburgh, N. Y.
Horne, H. W., New Hartford, Conn.
Houston, J. J. L...Jamaica, N. Y.
Hovey, O. E.....Plainfield, N. J.
Howard, L. T., Tompkinsville, N. Y.
Howard, O. Z.....New York City
Howe, C. E......Hastings, N. Y.
Howell, D. J...Washington, D. C.
Howell, W. A......Newark, N. J.
Howes, D. W.....Brooklyn, N. Y.
Hubbard, W. D., Brown Station, N. Y.
Hubbell, G. S.....New York City
Hudson, C. W....Montclair, N. J.
Hudson, H. W....New York City
Hughes, J. W..........Erie, Pa.
Hulsart, C. R.....New York City
Humphrey, R. L., Philadelphia, Pa.
Humphreys, D. C..Lexington, Va.
Hunt, C. A.......New York City
Hunt, Chas. Warren, New York City
Hunt, Robert W.....Chicago, Ill.
Hunt, W. H.......New York City
Hurd, H. L..White Plains, N. Y.
Hutchins, E. N....New York City
Hyde, J. L.......Westfield, Mass.

Ilsley, A. B....Washington, D. C.
Ives, A. S....Poughkeepsie, N. Y.
Ives, H. C.......Worcester, Mass.

James, A. R......Elizabeth, N. J.
Janes, G. P........Roselle, N. J.
Janvrin, N. H..Newburgh, N. Y.

Jarrett, E. S......New York City
Jenkins, J. E.....New York City
Johannesson, Sigvald,
Upper Montclair, N. J.
Johnson, D. C....Brooklyn, N. Y.
Johnson, L. E.......Steelton, Pa.
Johnson, T. S.....Granville, Ohio
Johnston, J. A..Springfield, Mass.
Jones, W. A...New Haven, Conn.
Just, G. A.,
Long Island City, N. Y.

Karner, W. J.....New York City
Kastenhuber, E. G., Jr.,
Beverly, N. J.
Kaufman, Gustave,
New York City
Keays, R. H....New Paltz, N. Y.
Keefer, C. H.,
Ottawa, Ont., Canada
Keith, H. C.......New York City
Kelley, W. D......Yonkers, N. Y.
Kershaw, W. H...New York City
Kimball, F. C......Summit, N. J.
King, P. S......Philadelphia, Pa.
Kinney, W. M....Pittsburgh, Pa.
Kinsey, W. A......Newark, N. J.
Kirchner, P. A....New York City
Kirkwood, H. C..Flushing, N. Y.
Knap, E. D.......New York City
Knap, J. M........Catskill, N. Y.
Knighton, J. A....New York City
Knox, S. B........New York City
Kornfeld, A. E....New York City
Krellwitz, D. W...New York City
Kuchar, F. M....Brooklyn, N. Y.

La Chicotte, H. A.,
Brooklyn, N. Y.
Lamb, W. A.......Helena, Mont.
Lamson, W. M....New York City
Lanagan, F. R.....Albany, N. Y.
Landreth, O. H.,
Schenectady, N. Y.
Langthorn, J. S...New York City
Lannan, L. E......New York City
Laut, F. P........New York City
Larmon, F. P...Cambridge, N. Y.
Larsson, C. G. E...New York City
Latey, H. N.......New York City
Latimer, C. A.....New York City
Latta, H. W.....Philadelphia, Pa.
Law, W. H.......Woodside, N. Y.
Lee, E. M........Paterson, N. J.
Leeper, J. B......Pittsburgh, Pa.
Leete, P. R...Short Beach, Conn.
Leeuw, H. A.....New York City
Leighton, George...Glenburn, Pa.
Leonard, H. R...Philadelphia, Pa.
Leser, Henry......New York City
Lesley, R. W....Philadelphia, Pa.
Letson, T. H.....New York City
Lewis, Clifford, Jr...Utica, N. Y.
Lewis, E. W...New Haven, Conn.
Lewis, J. O......Brooklyn, N. Y.
Lewis, N. P.......New York City
Lex, W. I.......Philadelphia, Pa.
Lichtner, W. O.,
Newton Highlands, Mass.
Lobo, Carlos.....Brooklyn, N. Y.
Loewenstein, Jacob,
New York City
Logan, W. S.......Newark, N. J.
Look, M. J..Brown Station, N. Y.
Loomis, Horace,
Mt. Vernon, N. Y.
Lowinson, Oscar...New York City
Lucas, E. W. V. C..New York City
Lucius, Albert.....New York City
Lueder, A. B......New York City
Lund, G. A.......New York City
Lyle, W. T...........Easton, Pa.
Lynde, Clifford....Walden, N. Y.

MacCracken, G. G..New York City
Macdonald, Charles,
Brooklyn, N. Y.
MacFeeters, J. O..New York City
MacGregor, R. A..New York City
Machen, H. B.....New York City
Mackenzie, K. G..Bayonne, N. J.
Macnab, G. T.....New York City

McCurdy, H. S. R.,
Brown Station, N. Y.
McGlashan, H. D..Berkeley, Cal.
McIntyre, W. A..Philadelphia, Pa.
McKenzie, T. H.,
Southington, Conn.
McKibbon, F. P.,
South Bethlehem, Pa.
McLure, N. R...Phœnixville, Pa.
McMinn, T. J....Brooklyn, N. Y.
McMullen, R. W..New York City
McNaugher, D. W.Pittsburgh, Pa.
Malmros, N. L....New York City
Malmros, N. L. A..Yonkers, N. Y.
Maltby, F. B......New York City
Manahan, E. G....New York City
Manley, L. B.,
West Roxbury, Mass.
Marple, W. McK...Scranton, Pa.
Marsh, A. L........Newark, N. J.
Marsh, F. B.......New York City
Marshall, R. A....Cranford, N. J.
Mason, Francis...New York City
Mazeau, Camille..Brooklyn, N. Y.
Mead, C. A.,
Upper Montclair, N. J.
Meadowcroft, William,
New York City
Meem, J. C......Brooklyn, N. Y.
Meggy, R. L. G...Fanwood, N. J.
Mehren, E. J......New York City
Melius, L. L.......New York City
Mercer, C. H........Steelton, Pa.
Merriman, Mansfield,
New York City
Merriman, Thaddeus,
Essex Fells, N. J.
Merritt, C. E........Utica, N. Y.
Meriwether, Coleman,
New York City
Metcalf, Leonard...Boston, Mass.
Middlebrook, C. T..Albany, N. Y.
Miles, G. F........New York City
Miller, H. A........Boston, Mass.
Miller, Hugh.....Potsdam, N. Y.
Miller, M. M......Yonkers, N. Y.
Miller, S. O.......New York City
Miller, Spencer.....New York City
Minor, E. E...New Haven, Conn.
Mitchell, C. H.,
Toronto, Ont., Canada
Mitchell, S. P...Philadelphia, Pa.
Modjeski, Ralph.....Chicago, Ill.
Mogensen, O. E..Plainfield, N. J.
Moisseiff, L. S.....New York City
Molitor, D. A......Detroit, Mich.
Molitor, Frederic...New York City
Möller, Louis,
Ft. William, Ont., Canada
Moore, E. J.......New York City
Moore, F. F....Hawthorne, N. Y.
Moore, W. H..New Haven, Conn.
Morrill, G. S....Dorchester, Mass.
Morrison, H. J...Prospect, N. Y.
Morrison, R. L....New York City
Morse, C. F.......Wallkill, N. Y.
Mott, W. E.......Pittsburgh, Pa.
Moyer, Albert.....New York City
Muir, A. W........Newton, N. J.
Muller, Leslie.....New York City
Munkelt, F. H....Brooklyn, N. Y.
Murphy, J. J......Detroit, Mich.
Myers, C. H......New York City

Nagle, J. C..College Station, Tex.
Najjar, S. A.....Brooklyn, N. Y.
Neely, J. T.,
Gatun, Canal Zone, Panama
Neely, W. R....New Paltz, N. Y.
Neff, F. H.......Cleveland, Ohio
Nelson, J. A......Paterson, N. J.
Neuhardt, Edwin..New York City
Neumeyer, R. E...Bethlehem, Pa.
Newcomb, W. T......Easton, Pa.
Newhall, H. L...New Paltz, N. Y.
Newman, A. T....New York City
Nichols, C. H..New Haven, Conn.
Noble, Alfred.....New York City
Norris, W. H.......Portland, Me.

Ockerson, J. A.....St. Louis, Mo.
O'Connell, G. P.,
Brown Station, N. Y.

Odell, F. S...Port Chester, N. Y.
Oestreich, H. L...Brooklyn, N. Y.
Ogden, J. C.......New York City
Okeson, W. R...Glen Ridge, N. J.
O'Rourke, J. F....New York City
Orrok, G. A......Brooklyn, N. Y.
Ott, S. J.......Rutherford, N. J.
Owen, James......Newark, N. J.
Owen, K. D......Montclair, N. J.
Oxholm, T. S.,
West New Brighton, N. Y.

Pagon, W. W.....Baltimore, Md.
Parker, C. J.......New York City
Parker, J. L.......New York City
Parsons, G. W.......Steelton, Pa.
Parsons, H. de B..New York City
Pawling, G. F...Philadelphia, Pa.
Payrow, H. G.,
Tufts College, Mass.
Peabody, W. W.,
White Plains, N. Y.
Pegram, G. H.....New York City
Pelz, C. E.........New York City
Pemoff, J. J.......New York City
Pendlebury, Edward,
Arlington, N. J.
Perkins, P. S....Providence, R. I.
Perkins, W. W. C.,
Niagara Falls, N. Y.
Perry, L. E........Salisbury, Md.
Philips, J. H...Glen Ridge, N. J.
Phillips, A. E..Washington, D. C.
Phillips, H. C.......Chicago, Ill.
Pierce, T. D.,
Cambridge Springs, Pa.
Pitcher, S. H....Worcester, Mass.
Pitkethly, D. T....Jamaica, N. Y.
Pohl, C. A........New York City
Polk, W. A........New York City
Poole, C. A......Rochester, N. Y.
Post, H. W.,
Mountain Lakes, N. J.
Potter, H. L.....Brooklyn, N. Y.
Powers, C. V. V...New York City
Pratt, A. H...White Plains, N. Y.
Pratt, M. D......Harrisburg, Pa.
Preston, C. H.....Norwich, Conn.
Preston, C. H., Jr.,
Waterbury, Conn.
Preston, H. L......Jordan, N. Y.
Price, F. O......Brooklyn, N. Y.
Price, P. L......Flushing, N. Y.
Prichard, H. S....Pittsburgh, Pa.
Priest, B. B...East Orange, N. J.
Purdy, S. M.....Brooklyn, N. Y.

Quimby, H. H..Philadelphia, Pa.
Quinn, M. F.......New York City

Raff, H. G........New York City
Randorf, C. A....Hamburg, N. Y.
Rankin, E. S......Newark, N. J.
Reeves, W. F......New York City
Reid, H. A........New York City
Reimer, A. A..East Orange, N. J.
Reimer, F. A..East Orange, N. J.
Remsen, T. R....Brooklyn, N. Y.
Reukauf, W. C...Brooklyn, N. Y.
Rice, G. S........New York City
Rich, M. S.....Washington, D. C.
Richardson, Clifford,
New York City
Richardson, J. H...Boston, Mass.
Richmond, J. P. W.,
Yonkers, N. Y.
Ridgway, Robert..New York City
Riegler, L. J.......Ben Avon, Pa
Rights, L. D......New York City
Ripley, Joseph.....Albany, N. Y.
Ripley, T. M......Fulton, N. Y.
Robbins, F. H.,
White Plains, N. Y.
Roberts, H. W.....New York City
Roberts, R. F......New York City
Robinson, E. W....New York City
Rogers, A. W......Buffalo, N. Y.
Rogers, E. H.,
West Newton, Mass.
Rogers, H. L......New York City

Rogge, J. C. L. . . . . New York City
Rohrer, Grant. . . . . . New York City
Rollins, J. W., Jr. . . Boston, Mass.
Roper, R. M. . . East Orange, N. J.
Rose, R. V. . . Niagara Falls, N. Y.
Rosenberg, F. . . . . Montclair, N. J.
Rossi, Irving. . . Jersey City, N. J.
Rudolph, W. E. . . Brooklyn, N. Y.
Rugg, W. F. . . . . . Peekskill, N. Y.
Ruggles, A. V. . . . . New York City
Rust, H. P. . . . . New York City
Ryan, M. H. . . . . . . New York City
Ryder, E. M. T. . . . Yonkers, N. Y.

Sanborn, F. B., Tufts College, Mass.
Sanborn, J. F. . . . Cornwall, N. Y.
Sanborn, M. F., Pleasantville, N. Y.
Sargent, P. D. . Washington, D. C.
Saville, Charles. . . . New York City
Sayles, R. W. . . . . . New York City
Schaeffer, Amos. . . . New York City
Schall, F. E., South Bethlehem, Pa.
Schermerhorn, H. O., Albany, N. Y.
Schneider, C. C. . Philadelphia, Pa.
Schreiber, J. M. . . . . Newark, N. J.
Schroeder, F. C. . . . New York City
Schwarze, C. T. . . . New York City
Seaman, H. B. . . . . New York City
Searight, G. P., White Plains, N. Y.
Searle, C. D. . . . . . . New York City
Seaver, Clifford, East Orange, N. J.
Senior, F. S. . . Montgomery, N. Y.
Serber, D. C. . . . . . . New York City
Shailer, R. A. . . . . . . Boston, Mass.
Shaughnessy, C. S., Cold Spring, N. Y.
Shaw, D. J. . . . . . . Katonah, N. Y.
Shaw, F. H. . . . . Lancaster, Pa.
Shaw, G. H. . . . . . Brooklyn, N. Y.
Shepherd, F. C. . . . . Boston, Mass.
Sheridan, L. V. . Indianapolis, Ind.
Sherman, A. L., White Plains, N. Y.
Sherman, C. W. . . . . Boston, Mass.
Sherrerd, J. M. . . . . Easton, Pa.
Shoemaker, M. N. . Newark, N. J.
Sickman, A. F. . . . Holyoke, Mass.
Sikes, Z. H. . . . . . . Yonkers, N. Y.
Simpson, G. F. . . . New York City
Sitt, W. T. . . . . New York City
Skillin, E. S. . . . . . New York City
Skinner, F. W. . . . . New York City
Skinner, J. F. . . . Rochester, N. Y.
Sloan, W. G. . . . . . . New York City
Slocum, C. M. . . Springfield, Mass.
Smith, A. C., Jr. . . Baltimore, Md.
Smith, Augustus. . . Bayonne, N. J.
Smith, C. E., West New Brighton, N. Y.
Smith, C. H. . . . Middletown, N. Y.
Smith, C. W. . . . . . . New York City
Smith, J. R. . . . . New York City
Smith, J. W. . . . . . . New York City
Smith, M. H. . . . . . New York City
Smith, W. F. . . . . . Valhalla, N. Y.
Smith, W. M., West New Brighton, N. Y.
Smoley, C. K. . . . . . . Scranton, Pa.
Snell, H. B. . . . . . . Brooklyn, N. Y.
Snow, J. B. . Richmond Hill, N. Y.
Snow, J. P., West Somerville, Mass.
Snyder, C. H. . . . . . Oswego, N. Y.
Snyder, F. A. . . . . . New York City
Snyder, G. D. . . . . . New York City
Soest, H. C. . . . . . . New York City
Soper, G. A. . . . . . . New York City
Spaulding, F. A. . Pittsfield, Mass.
Spear, W. E. . . . . . . New York City
Spencer, C. B. . . . . New York City
Splitstone, C. H., East Orange, N. J.
Spofford, C. M. . . . . . Boston, Mass.
Spooner, A. N. . . New Dorp, N. Y.

Stanford, H. R., Washington, D. C.
Staniford, C. W. . . New York City
Sprague, E. L., Jr. . Valhalla, N. Y.
Stark, C. W. . . . . . . New York City
Starr, F. C. . . . . Washington, D. C.
Starr, H. H. . . . . . . New York City
Stearns, F. L. . . . . . New York City
Stearns, F. P. . . . . . . Boston, Mass.
Stearns, R. H. . . . Hartford, Conn.
Steffens, W. F. . . . . . Boston, Mass.
Stehle, F. C. . . . Mt. Vernon, N. Y.
Stephens, A. W., East Orange, N. J.
Stephenson, F. H. . New York City
Stern, E. W. . . . . . . New York City
Stevens, H. C. . . . . New York City
Stevenson, W. F., New Rochelle, N. Y.
Stewart, J. M. . . . . New York City
Stoddard, G. C. . . . New York City
Stoddard, R. F. . Bridgeport, Conn.
Stone, W. W. . . . Newburgh, N. Y.
Stott, H. G. . . . . New York City
Strachan, Joseph . . Brooklyn, N. Y.
Strawn, T. C. . . . . . New York City
Strickler, F. W., Mt. Vernon, N. Y.
Strong, J. B. . . . . . Hillburn, N. Y.
Strouse, W. F. . . . . Baltimore, Md.
Sullivan, J. F. . . . . New York City
Sutton, Frank . . Washington, D. C.
Swaab, S. M. . . . Philadelphia, Pa.
Swain, G. F. . . . . Boston, Mass.
Swezey, E. C. . . . Brooklyn, N. Y.
Swindells, J. S. . . Brooklyn, N. Y.

Taber, G. A. . . . . . Brooklyn, N. Y.
Taft, J. R. . . . . New York City
Taggart, R. C. . . . Brooklyn, N. Y.
Tait, Harold, Long Island City, N. Y.
Talbot, A. N. . . . . Urbana, Ill.
Taylor, C. F. . . . . New York City
Taylor, G. McC. . Red Bank, N. J.
Tays, E. A. H., San Blas, Sinaloa, Mexico
Temple, W. H. G. Providence, R. I.
Tenney, W. R. . . . Brooklyn, N. Y.
Terry, J. H. . . . . Philadelphia, Pa.
Thacher, Edwin . . New York City
Thayer, N. A. . . . New York City
Thoma, Jacob, Long Island City, N. Y.
Thomas, C. D. . . . Brooklyn, N. Y.
Thomes, E. H. . . . . Jamaica, N. Y.
Thompson, J. A. . New York City
Thompson, S. C. . New York City
Thompson, S. E., Newton Highlands, Mass.
Thomson, Alexander, Jr., Walden, N. Y.
Thomson, John . . . New York City
Thomson, S. F. . New Paltz, N. Y.
Thomson, T. K. . . New York City
Thurber, C. D. . . . Brooklyn, N. Y.
Tidd, A. W. . . White Plains, N. Y.
Tighe, J. L. . . . . . . Holyoke, Mass.
Tillson, G. W. . . . Brooklyn, N. Y.
Tilt, G. E. . . . . New York City
Timberlake, S. M., White Plains, N. Y.
Tinker, C. A. . . . . Westfield, Mass.
Tompkins, E. DeV. . Pelham, N. Y.
Torrey, J. E. . . . . . Paterson, N. J.
Tower, J. W. . . . . New York City
Tozzer, A. C. . . . . New York City
Trautwine, J. C., Jr., Philadelphia, Pa.
Tretter, G. A. . . . . Roanoke, Va.
Tribus, L. L. . . . . . New York City
Triest, W. G. . . . . New York City
Trout, C. E. . . . . . New York City
Trow, F. H. . Brown Station, N. Y.
Trowbridge, D. S. . New York City
True, A. O. . . . . Albany, N. Y.
Tull, R. W. . . . . . . Fanwood, N. J.
Turner, D. L. . . . . . New York City
Turner, E. K. . . . . . . Boston, Mass.
Tuska, G. R. . . . . New York City

Tuttle, A. S. . . . . . New York City
Tyler, W. R. . . . . New York City

Upson, M. M. . . . Englewood, N. J.

Valleley, W. P. . . New York City
Van Cleve, A. H. . New York City
Van Horne, J. G. New York City
Van Norden, E. M., Brooklyn, N. Y.
Van Winkle, Edward, New York City
Vaughan, L. B. . . Kingston, N. Y.
Verveer, E. L. . . . . New York City
Vincent, J. I. . . . . New York City
Vrooman, Morrell, Gloversville, N. Y.

Wachter, C. L. . . . New York City
Waddell, F. C., West New Brighton, N. Y.
Wagner, J. C. . . . Philadelphia, Pa.
Wait, B. H. . . . . . . New York City
Waite, G. B. . . . . . . New York City
Walker, E. L. . . . . New York City
Wallace, J. F. . . . . New York City
Ward, C. D. . . . . . . New York City
Wardle, E. B. . West Nutley, N. J.
Warnock, W. H. . . New York City
Watkins, F. W., White Plains, N. Y.
Watt, D. A. . . . . . . New York City
Watters, G. L., South Bethlehem, Pa.
Webster, G. S. . . Philadelphia, Pa.
Wegmann, Edward, New York City
Wells, C. E. . . White Plains, N. Y.
Wells, G. E. . . . Naugatuck, Conn.
Wendt, E. F. . . . . . Pittsburgh, Pa.
Wentworth, C. A. . . . . Wayne, Pa.
Wentworth, G. L. . Yonkers, N. Y.
Weston, F. S. . . Middleboro, Mass.
Weston, R. S. . . . . . Boston, Mass.
Whinery, Samuel, East Orange, N. J.
Whipple, G. C. . . . New York City
Whiskeman, J. P. . New York City
White, Lazarus . . . New York City
White, T. S. . . . . Beaver Falls, Pa.
Whitney, F. O. . . . . . Boston, Mass.
Whittemore, J. O., Mt. Vernon, N. Y.
Wiggin, E. W. . New Haven, Conn.
Wiggin, T. H. . . . Scarsdale, N. Y.
Wilcock, Frederick, Brooklyn, N. Y.
Wilcox, Frank. . . . Pittsburgh, Pa.
Wild, H. J. . . . . Chester, Pa.
Wildes, W. G. . . . Rochester, N. Y.
Wiley, W. H. . East Orange, N. J.
Williams, E. G. . . . New York City
Williams, F. P., Mechanicsville, N. Y.
Williams, Frederick, New London, Conn.
Williams, G. S. . Ann Arbor, Mich.
Willis, A. J. . . . . . New York City
Wills, W. S. . . . . Neligh, Nebr.
Wilmot, James. . . . New York City
Wilmot, Sydney. . New York City
Wilson, C. W. S., New Rochelle, N. Y.
Wilson, H. M. . . . . Pittsburgh, Pa.
Wilson, P. H. . . . Philadelphia, Pa.
Wilson, T. L. . . . . . New York City
Wilson, W. T. . . . . New York City
Wilson, William. . New York City
Winsor, F. E. White Plains, N. Y.
Winsor, G. A. . . . . Valhalla, N. Y.
Winsor, H. D. . . . . New York City
Wise, C. R. . . . . Passaic, N. J.
Witmer, F. P. . East Orange, N. J.
Wolcott, C. S. . . . . . Hornell, N. Y.
Wolfe, F. G. . . . . Scranton, Pa.
Wolff, A. D., Jr. . . . New York City
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Wood, G. P. . . . . . Peekskill, N. Y.

Wood, H. S......New York City
Wood, I. S......Providence, R. I.
Woodard, S. H..Scarsdale, N. Y.
Worthington, Charles, New York City
Wright, J. B......New York City
Wyckoff, C. R., White Plains, N. Y.
Wyman, A. M....New York City

Yappan, Adolph......Chicago, Ill.
Yates, P. K......Brooklyn, N. Y.
Yates, W. H.......Albany, N. Y.
Yereance, W. B...New York City
Zook, M. A......Plainfield, N. J.

PROGRESS REPORT OF SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

I. Introduction.

1.—Appointment and Work of Committee.

In 1903 and 1904 Special Committees were appointed by the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers, for the purpose of investigating current practice and providing definite information concerning the properties of concrete and reinforced concrete and to recommend necessary factors and formulas required in the design of structures in which these materials are used. The history of the appointment of the committees is as follows:

At the Annual Convention of the American Society of Civil Engineers, held at Asheville, N. C., June 11th, 1903, the following resolution was adopted:

"It is the sense of this meeting that a Special Committee be appointed to take up the question of concrete and steel-concrete, and that such committee co-operate with the American Society for Testing Materials, and the American Railway Engineering and Maintenance of Way Association."

Following the adoption of this resolution, a Special Committee on Concrete and Steel-concrete was appointed by the Board of Direction on May 31st, 1904. At the Annual Meeting, held January 18th, 1905, the title of this Special Committee was, at the request of the Committee, changed to "Special Committee on Concrete and Reinforced Concrete." This Special Committee held its first meeting at Atlantic City, N. J., June 17th, 1904, and effected an organization; Mr. C. C. Schneider was appointed Chairman and Mr. J. W. Schaub, Secretary. Mr. Schneider resigned from the Committee on January 3d, 1911, and the Board of Direction, on January 31st, 1911, appointed Mr. J. R. Worcester as Chairman. On the resignation of Mr. J. W. Schaub, Mr. Richard L. Humphrey was appointed Secretary on October 11th, 1905.

At the first meeting of the Committee it was decided to co-operate with similar committees which had been appointed by the American Society for Testing Materials and the American Railway Engineering and Maintenance of Way Association through the organization of a Joint Committee on Concrete and Reinforced Concrete.

Subsequent meetings of the Committee were held at New York in January, 1906, 1907, 1909, 1910, 1911, and 1912; on December 10th,

1907; on March 1st, April 3d, May 1st, 1911, and on November 20th, 1912; at Philadelphia in December, 1908; and at Chicago in June, 1910. The Committee was also represented at all the meetings of the Joint Committee.

At the annual meeting of the American Society for Testing Materials, held on July 1st, 1903, at the Delaware Water Gap, the following resolution was unanimously adopted:

"That the Executive Committee be requested to consider the desirability of appointing a committee on 'Reinforced Concrete', with a view of co-operating with the committees of other societies in the study of the subject."

At the meeting of the Executive Committee of the American Society for Testing Materials, held on December 5th, 1903, a special committee on "Reinforced Concrete" was appointed.

The American Railway Engineering and Maintenance of Way Association appointed a Committee on Masonry on July 20th, 1899, with instructions, as a part of its duties, to prepare specifications for concrete masonry. A preliminary set of specifications for Portland cement concrete was reported to and adopted by the Association on March 19th, 1903. At the meeting held in Chicago on March 17th, 1904, the Committee on Masonry was authorized to co-operate with the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, and, following this action, a special sub-committee was appointed.

At a meeting of the several special committees representing the above-mentioned societies, held at Atlantic City, N. J., on June 17th, 1904, arrangements were completed for collaborating the work of these several committees through the formation of the Joint Committee on Concrete and Reinforced Concrete. Mr. C. C. Schneider was elected temporary chairman and Professor A. N. Talbot temporary secretary. The proposed plan of action of the Special Committee of the American Society of Civil Engineers was outlined, involving the appointment of sub-committees on Plan and Scope, on Tests, and on Ways and Means.

The Joint Committee, at its first meeting, invited the Association of American Portland Cement Manufacturers to join in its deliberations through a committee appointed for the purpose.

The Joint Committee, at meetings at St. Louis in October, 1904, and at New York in the following January, perfected its organization by the adoption of rules and the choice of Mr. C. C. Schneider as Chairman, Mr. Emil Swensson, Vice-Chairman, and Mr. J. W. Schaub, Secretary. Later, on the resignation of Mr. Schaub, Mr. Richard L. Humphrey was chosen Secretary. Sub-committees on Plan and Scope, on Tests, and on Ways and Means were appointed.

The Joint Committee, as thus organized, consisted of the following members:

OFFICERS.

Chairman.—C. C. SCHNEIDER.
Vice-Chairman.—EMIL SWENSSON.
Secretary.—RICHARD L. HUMPHREY.

MEMBERS.

American Society of Civil Engineers (Special Committee on Concrete and Reinforced Concrete):

Greiner, J. E., Consulting Engineer, Baltimore and Ohio Railroad, Baltimore, Md.

Hatt, W. K., Professor of Civil Engineering, Purdue University, Lafayette, Ind.

Hoff, Olaf, Vice-President, Butler Brothers, Hoff and Company, New York, N. Y.

Humphrey, Richard L., Consulting Engineer; Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.

Lesley, R. W., President, American Cement Company, Philadelphia, Pa.

Schaub, J. W., Consulting Engineer, Chicago, Ill.

Schneider, C. C., Consulting Engineer, Philadelphia, Pa.

Swensson, Emil, Consulting Engineer, Pittsburgh, Pa.

Talbot, A. N., Professor of Municipal and Sanitary Engineering, in Charge of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

Worcester, J. R., Consulting Engineer, Boston, Mass.

American Society for Testing Materials (Committee on Reinforced Concrete):

Fuller, William B., Consulting Engineer, New York, N. Y.

Heidenreich, E. Lee, Consulting Engineer, New York, N. Y.

Humphrey, Richard L., Consulting Engineer; Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.

Johnson, Albert L., Consulting Engineer, St. Louis, Mo.

Lanza, Gaetano, Professor of Theoretical and Applied Mechanics, Massachusetts Institute of Technology, Boston, Mass.

Lesley, R. W., President, American Cement Company, Philadelphia, Pa.

Marburg, Edgar, Professor of Civil Engineering, University of Pennsylvania, Philadelphia, Pa.

Mills, Charles M., Principal Assistant Engineer, Philadelphia Rapid Transit Company, Philadelphia, Pa.

Moisseiff, Leon S., Engineer of Design, Department of Bridges, New York, N Y.

Quimby, Henry H., Assistant Engineer of Bridges, Bureau of Surveys, Philadelphia, Pa.

Taylor, W. P., Engineer in Charge of Testing Laboratory, Philadelphia, Pa.

Thompson, Sanford E., Consulting Engineer, Newton Highlands, Mass.

Turneaure, F. E., Dean of College of Mechanics and Engineering, University of Wisconsin, Madison, Wis.

Wagner, Samuel Tobias, Assistant Engineer, Philadelphia and Reading Railroad, Philadelphia, Pa.

Webster, George S., Chief Engineer, Bureau of Surveys, Philadelphia, Pa.

American Railway Engineering Association (Sub-Committee on Reinforced Concrete):

Beckwith, Frank, Engineer of Bridges and Structures, Lake Shore and Michigan Southern Railroad, Cleveland, Ohio.

Boynton, C. W., Inspecting Engineer, Cement Department, Illinois Steel Company, Chicago, Ill.

Cunningham, A. O., Chief Engineer, Wabash Railroad, St. Louis, Mo.

Scribner, Gilbert H., Jr., Contracting Engineer, Chicago, Ill.

Swain, George F., Professor of Civil Engineering, Massachusetts Institute of Technology, Boston, Mass.

Association of American Portland Cement Manufacturers (Committee on Concrete and Steel Concrete):

Fraser, Norman D., President, Chicago Portland Cement Company, Chicago, Ill.

Griffiths, R. E., Vice-President, American Cement Company, Philadelphia, Pa.

Hagar, Edward M., Manager, Cement Department, Illinois Steel Company, Chicago, Ill.

Newberry, Spencer B., Manager, Sandusky Portland Cement Company, Sandusky, Ohio.

Since organization, the following changes have occurred in the *personnel* of the Joint Committee:

J. W. Schaub, died March 30th, 1909.
C. C. Schneider, resigned January 3d, 1911.
Ernest R. Ackerman, resigned.
T. J. Brady, resigned.
Frank Beckwith, resigned.
A. O. Cunningham, resigned.
George F. Swain, resigned.

The following representatives of the American Railway Engineering Association have since been appointed:

Thompson, F. L., Engineer of Bridges and Buildings, Illinois Central Railroad, Chicago, Ill.

Alternates:

Hotchkiss, L. J., Assistant Bridge Engineer, Chicago, Burlington and Quincy Railroad, Chicago, Ill.

Prior, J. H., Assistant Engineer, Chicago, Milwaukee and St. Paul Railway, Chicago, Ill.

Schall, F. E., Bridge Engineer, Lehigh Valley Railroad, South Bethlehem, Pa.

Tuthill, Job, Assistant Engineer, Cincinnati, Hamilton and Dayton Railway, Cincinnati, Ohio.

At a meeting of the Joint Committee held at Atlantic City, N. J., on June 30th, 1911, Mr. J. R. Worcester was elected Chairman.

Meetings of the Joint Committee have been held as follows:

June 17th, 1904, at Atlantic City, N. J.
Oct. 4th, 5th, 6th, 1904, at St. Louis, Mo.
Jan. 17th, 1905, at New York, N. Y.
June 21st, 1905, at Cleveland, Ohio.
June 30th, 1905, at Atlantic City, N. J.
Oct. 11th, 1905, at New York, N. Y.
Dec. 14th, 1905, at New York, N. Y.
June 21st, 1906, at Atlantic City, N. J.
Dec. 13th, 1906, at New York, N. Y.
Jan. 15th, 1907, at New York, N. Y.
March 7th, 1907, at New York, N. Y.
March 18th, 1907, at Chicago, Ill.
June 21st, 22d, 1907, at Atlantic City, N. J.
Dec. 10th, 1907, at New York, N. Y.
Oct. 27th, 28th, 1908, at New York, N. Y.
Dec. 9th, 10th, 11th, 1908, at Philadelphia, Pa.
June 30th, 1911, at Atlantic City, N. J.
Nov. 20th, 1912, at New York, N. Y.

At the meeting of the Joint Committee at St. Louis in October, 1904, it was determined to arrange tests at such technological institutions as were provided with the requisite facilities and were willing to co-operate, the Committee, through its sub-committee on Ways and Means, to provide materials, and through its sub-committee on Tests, to consult as to lines of testing and to advise as to methods. The following ten institutions, Case School of Applied Science, Columbia University, Cornell University, University of Illinois, State University of Iowa, Massachusetts Institute of Technology, University of Minne-

sota, Ohio State University, Purdue University, and University of Wisconsin, undertook a preliminary series of tests, and carried them through, in due time reporting their results to the Committee.

Through the inability of the Committee to do as much as it had hoped by way of furnishing uniform materials for these tests and exercising a proper supervision, the results were not as serviceable as they would have been if the full plans had been carefully carried out; but much important information was received in this manner, and the Committee desires to express its gratitude to the professors and students who assisted so kindly in this work.

The results were collated and edited by the Secretary of the Committee at the Structural Materials Testing Laboratories of the U. S. Geological Survey, St. Louis, and the results, in typewritten form, were circulated among the members of the Committee. It was hoped that they might be published by the Geological Survey as a Bulletin, but in that the Committee was disappointed, though some of the results have been published in bulletins and papers issued by their authors.

In June, 1905, the U. S. Geological Survey proposed to co-operate with the Joint Committee to the extent of placing the tests made at the St. Louis Laboratory at its service and allowing the Committee the privilege of advising as to what tests of concrete and reinforced concrete should be conducted there. This co-operation was welcomed by the Committee, and was brought about by the fact that the Secretary of the Committee, who was also the Chairman of the sub-committee on tests, was in charge of the St. Louis Laboratory.

During the five years in which the investigations of structural materials were in progress under the direction of the United States Geological Survey, a large amount of data relating to concrete and reinforced concrete was obtained. These investigations have included the survey of the constituent materials of concrete, such as sands, gravels, and crushed stone, in the various parts of the United States, covering their strength as mortars or concretes in various consistencies and proportions.

A number of series of tests of plain and reinforced concrete beams was made, covering the influence of character of aggregates, proportions and age, percentage of reinforcement, the effect of the variation in span relative to the depth, methods of anchorage of the reinforcement, etc., upon strength. A study was made of the effect of the personal equation in tests of beams, made by three construction companies operating in St. Louis, and by the employees of the testing laboratory. Tests covering bond, shear, compressive strength, and weight per cubic foot, for various classes of aggregates, were made.

Among other investigations were tests of reinforced concrete slabs of 12 ft. span, supported at two and four edges, strength and other properties of cement hollow building blocks, of the permeability of

cement mortars and concretes, value of various water-proofing and damp-proofing preparations, effect of alkali and sea water on cement mortars and concretes, the fire-resistive properties of concrete and other structural materials, and these have been made and published, in part.

The collation and study of the data obtained were seriously handicapped through lack of funds available for this purpose, the large part of the appropriation being devoted to work urgently required by the various Government Bureaus. Of the annual Government appropriation of $100 000, there was never available more than $15 000 per annum for the investigation of concrete and reinforced concrete, and for several years the amount did not exceed $5 000 a year. None of this was available for the publication of results, and the allotment from the funds provided for all Government printing was wholly inadequate for the purpose.

On June 30th, 1910, Congress transferred this work to the Bureau of Standards, together with the data collected. It is understood that arrangements have been made by which the data of the tests will be published as rapidly as conditions permit.

The Committee has had the benefit of the results of investigations by a number of laboratories, some of which were under the direct supervision of its members. The extent and varied character of the tests, and their interpretation by those in charge, made them of especial value to the Committee.

The Committee also has had the advantage of investigations made in foreign laboratories.

At a meeting of the Joint Committee at Atlantic City, on June 30th, 1905, it was decided to divide among its members the work of collating and digesting the results of all available tests on concrete and reinforced concrete, and, in pursuance of this resolution, sub-committees were appointed on the following subjects:

Historical.
Aggregates, Proportions and Mixing.
Physical Characteristics, Water-proofing, etc.
Strength and Elastic Properties.
Simple Reinforced Concrete Beams.
T-Beams, Floor Slabs, etc.
Columns and Piles.
Fire-resistive Qualities.
Failures of Concrete Structures.
Arches.

A large amount of work was done by these sub-committees and extensive reports were submitted by most of them. These reports were typewritten in manifold and circulated among the members of

the Joint Committee, and were of great value to the Committee in arriving at its conclusions.

The Sub-Committee on Ways and Means raised by subscription about $8 000, which was used for preliminary investigations and expenses incident to printing its report and carrying on the work of the Committee. The Committee desires to express its appreciation for contributions and for donations of materials.

Even with this support, the field of activity of the Committee has been limited in scope, and it has been unable to undertake investigations of its own.

In 1908 the Committee began the preparation of the Progress Report which was submitted to the Society in January, 1909. A preliminary outline was prepared by the Secretary and submitted to the Committee in October. On October 27th, a meeting of the Joint Committee was held at New York, at which the report was discussed paragraph by paragraph, and chapters were referred to sub-committees and carefully revised during the following three weeks. The whole, as thus amended and revised, was again submitted in print to a full meeting held at Philadelphia, on December 9th, 10th, and 11th, and again was gone over in great detail. As a result of those two meetings, a considerable amount of matter, which it was at first intended to include, was omitted on account of slight disagreements as to its form and lack of time to work it into satisfactory shape, and to this fact may be attributed some of the criticisms which have been elicited It is hoped, in this report, to avoid these objections.

In the spring of 1911 the work of revising the 1909 Progress Report was taken up, and a number of meetings were held. The discussions submitted to the American Society of Civil Engineers and subsequent papers relating to the same subject were carefully considered, and differences of opinion between members of the Committee were threshed out.

Through the co-operation of the societies represented on the Joint Committee, the report was again put in type, and the necessary editions were printed for the use of the members of the Committee, the last bearing the date of August 1st, 1911. In the form thus reached, the report remained until November 20th, 1912, when the Committee again met in New York and gave the final review needed to bring it into the shape in which it is now presented.

2.—Historical Sketch of Use of Concrete and Reinforced Concrete.

In considering the history of concrete and reinforced concrete, a distinction should be made between the two. The use of concrete extends back to long before the Christian era—on the other hand, the art of reinforced concrete is in its infancy.

The use of concrete by the ancient Romans was due to the discovery of the fact that volcanic ash or puzzolan, when mixed with slaked lime, made a cement possessing hydraulic properties. The durability of this work of the Romans was due largely to favorable climatic conditions and the character of the cement used.

From the downfall of the Roman Empire to the last half of the Eighteenth Century the manufacture of cement seems to have been discontinued. The Roman cement mortars and concretes surviving the ravages of the elements became so hard that the cement acquired a reputation that led the early experimenters of the Eighteenth Century to seek to recover this supposedly lost Roman art. Evidently, no concrete was used during this period, for the necessity of simultaneous induration in the interior and exterior of the mass prevents the use of lime alone in concrete, and requires the use of some material having hydraulic qualities. This fact limited the use of concrete to regions where hydraulic limes and cements were to be found.

In 1756 Smeaton discovered that an argillaceous limestone produced a lime that would set and harden under water, but no immediate appreciation of this knowledge appears to have resulted.

Natural cement was first manufactured by Parker in 1795, as a result of an attempt to equal or excel Roman cement, and in 1796 he took out an English patent. Natural cement was first produced in America in 1818, and for a long time was the principal cement used. With the introduction of Portland cement, and the reduction in the cost of manufacture, there has been a gradual substitution of Portland for natural cement. The production of natural cement reached a maximum of nearly 10 000 000 bbl. in 1899 and gradually decreased to about 9 000 000 bbl. in 1911.

The art of manufacturing Portland cement was discovered in 1811 by Joseph Aspdin, and patented by him in 1824. He called this cement "Portland" by reason of its resemblance to a building stone obtained from the Isle of Portland, off the coast of England. Up to 1850 very little progress was made in the manufacture of this cement in England. Since 1855, however, the increase in the production in Europe has been steady, and its superiority has led to a gradually increasing use in such structures as require concrete in mass, as foundations, fortifications, sea-walls, docks, locks, etc. While Portland cement was first manufactured in 1824, and was produced in 1871 by David O. Saylor, at Coplay, Pa., and by Thomas Millen, at South Bend, Ind., it was not until the early Eighties that it was manufactured to any extent in America. From that time on, the production has rapidly increased, reaching the enormous total of nearly 80 000 000 bbl. in 1911. This increase in production has been largely stimulated by the reduction in cost of Portland cement through the perfection of the American

methods, the introduction of reinforced concrete, and the extensive use of cement during the last few years.

In 1850 Joseph Gibbs obtained a British patent for casting solid walls in wooden moulds, and in 1897 C. W. Stevens obtained a patent for making artificial cast stone with concrete. It is not clear, however, that these inventors were the first to use the material in a similar way.

The origin of the idea of increasing the load-carrying capacity of concrete by reinforcing it with metal embedded in it is generally attributed to Joseph Monier, a French gardener, who used a wire frame or skeleton embedded in concrete in the construction of flower pots, tubs, and tanks, in 1867, and for which he obtained the first patent of the kind in the same year. This was not the first use of the material, however, as Lambot constructed a boat of reinforced concrete in 1850, which was exhibited at the Paris Exposition in 1853. He took out an English patent in 1855.

In France, in 1861, François Coignet applied the principles of reinforced concrete in the construction of beams, arches, pipes, etc., and with Monier exhibited some of their work at the Paris Exposition of 1867. Coignet also took out an English patent in 1855. In England, in 1854, W. B. Wilkinson took out a patent for a reinforced concrete floor. In America, Ernest L. Ransome used metal in combination with concrete as early as 1874, and W. E. Ward erected, in 1875, at Port Chester, New York, a house built entirely of reinforced concrete.

Monier, while not the first to apply it, obtained the first patents for reinforced concrete, the German and American rights of which he disposed of to G. A. Wayss and Company in 1880. Wayss and J. Bauschinger, shortly after, began the tests on this material which were published in 1887.

Thaddeus Hyatt, an American engineer, employed David Kirkaldy of London to make the experiments on reinforced concrete which Hyatt published in 1877. The theories of Hyatt were applied in a practical way to building construction in 1877 by H. P. Jackson, of San Francisco.

In America Ransome, between 1874 and 1884, constantly increased his application of metal reinforcement consisting of old wire rope and hoop iron, gradually realizing the necessity for using it with a greater regard for its proper position in the mass, and in 1884 took out the first patent for a deformed bar. Prior to this, reinforced concrete was used but little in the United States. Ransome built his first important structure in 1890, the Leland Stanford, Jr., Museum Building, 312 ft. long, two stories high, with basement, the walls and floors of which were of reinforced concrete. Since 1891, when the first slabs of reinforced concrete were used in America, the development has been rapid.

The introduction of this form of construction proceeded more

slowly in Europe, and between 1891 and 1894 Moeller in Germany, Wünsch and Emperger in Hungary, Melan in Austria, and Hennebique in France, were pioneers in its development. Hennebique built reinforced concrete slabs as early as 1879, but did not patent his system of construction until 1892.

The first published method of computation was by Koenen and Wayss in 1886. Subsequent theories have been advanced by de Mazas, Neuman, Melan, Coignet, de Tedesco, Von Thullie, Ostenfeld, Sanders, Spitzer, Lutken, Ritter, Hatt, Talbot, Turneaure, and others. As early as 1884, Ransome worked out methods of calculation independent of other investigators, and in 1899 Considère published his important series of tests from which he deduced his methods of calculation.

During the last ten years the earlier theories have been somewhat modified as experience has been gained and as the fund of experimental knowledge has accumulated. The trend of the modifications has been toward greater harmony in methods of calculation. Some of the earlier assumptions have been proved fallacious, and generally abandoned. On the other hand, some of the refinements of calculation, though known to be in accordance with facts, have, by general consent, been discarded, as they do not affect the design materially, or are taken into account by a modification of the constants. Among these are the value of the concrete in the tension side of a beam and the lack of a uniform modulus of elasticity in compression of concrete under widely varying stress. The earlier theories did not deal with the diagonal tension under shearing stresses. This has been found to be a most important consideration, and much attention has been paid to it in recent years. In spite of the study which has already been given to it, however, there is still much to learn in this direction. The action of various forms of reinforcement in columns has received much consideration, and there is still a wide difference of opinion as to the efficacy of some forms of column reinforcement. Many experiments have been made in this branch of the subject, and practice appears to be gradually converging toward greater uniformity.

In the preparation of this historical sketch, the Committee has endeavored to verify the facts, and has received the co-operation of H. Kempton Dyson, Secretary of the Concrete Institute of England, Alfred Huser, President, Deutscher Beton-Verein, C. von Bach, Otto Leube, of Germany, Karl Naehr, of Austria, Joseph Schustler, of Hungary, and H. I. Hannover, of Denmark, to whom the Committee wishes to acknowledge its appreciation and thanks.

3.—Authorities on Which Recommendations are Based.

It has been suggested that a report such as this should include all the data on which conclusions are based. The impracticability of this may not be realized by those who are not familiar with the enormous

quantity of matter involved. There are, however, reasons other than the magnitude of the task which tend to show that full publication is not advisable. One of these is that most of the experimental results have already appeared in print and are now available, and a reprint of them would be of no great advantage to any one. Where originally printed they are frequently accompanied with comments and deductions by their authors, which are of great value as such but could scarcely be copied by the Committee. Another reason against publication is that, in the large part of the experimental work consulted, it has been found that certain vitally important information, either with regard to the materials, the way in which they are manipulated, or as to the precise results reached, is lacking. The omission of measurements of deformations, of course, frequently renders results of little value. While such tests may have some use on account of particular facts developed, a large part may be useless, and consequently unsuitable for publication. The difficulty of separating the valuable from the valueless would be almost insurmountable.

It may not be improper, however, to append the following list of authors and references, as comprising a considerable part of the most important published material on the subject under consideration:

C. v. Bach.—"Compressive Tests": *Deutsche Bauzeitung,* 1905, 68 (No. 17). *Mitteilungen über Forschungsarbeiten,* Nos. 22, 29, 39, 45-47, 72-74.

E. Candlot.—"Cements and Mortars": *"Ciments et Chaux Hydrauliques,"* 1898, pp. 446, 447.

Howard A. Carson.—"Plain and Reinforced Concrete Beams": Boston Transit Commission, 10th Annual Report, 1904, Appendix G.

A. Considère.—"Reinforced Beams and Columns": *Comptes Rendus de l'Academie des Sciences,* CXXVII, p. 992; CXXIX, p. 467; CXXXV, Sept. 8, 1902; CXL, June 30, 1905.

F. v. Emperger.—*"Forschungsarbeiten auf dem Gebiete des Eisenbetons,"* No. 8.

R. Feret.—*"Sur la Compacité des Mortiers Hydrauliques": Annales des Ponts et Chaussees,* 1892, II. "Composition, Various Tests of Reinforcing": *Etude Experimentale du Ciment Armé,* 1906.

William B. Fuller and Sanford E. Thompson.—"Composition and Density": *Transactions,* American Society of Civil Engineers, Vol. LIX, 1907, p. 67.

William K. Hatt.—"Reinforced Concrete Beams": *Proceedings,* American Society for Testing Materials, Vol. II, 1902, p. 161; *Journal,* Western Society of Engineers, June, 1904.

James E. Howard.—"Watertown Arsenal Tests of Cubes and Reinforced Columns": Tests of Metals, U. S. A., 1897, 1898, 1899, 1903, 1905, and 1906; *Proceedings,* American Society for Testing Materials, Vol. VI, 1906, p. 346.

Richard L. Humphrey.—"St. Louis Laboratory Tests of Aggregates, Beams, Prisms, Fire-Resistance": U. S. Geological Survey, *Bulletins, 324, 329, 331, 344, 370;* and Bureau of Standards, Technologic Paper, 2.

George A. Kimball.—"Compressive Tests of Cubes": Tests of Metals, U. S. A., 1899.

Gaetano Lanza.—"Reinforced Columns and Beams": *Transactions,* American Society of Civil Engineers, Vol. L, 1903, p. 483; *Proceedings,* American Society for Testing Materials, Vol. VI, 1906, p. 416.

Elmer J. McCaustland.—"Plain and Reinforced Columns": *Engineering News,* Vol. LIII, p. 614, June 15, 1905.

Edgar Marburg.—"Reinforced Concrete Beams and Piers": *Proceedings,* American Society for Testing Materials, Vol. IV, 1904, p. 508; Vol. IX, 1909, p. 509.

E. Mörsch.—*"Der Eisenbetonbau."*

Charles L. Norton.—"Fire-proofing, Protection of Steel by Concrete": Boston Insurance Engineering Experiment Station Reports, IV and IX.

Logan W. Page.—"Properties of Oil-mixed Portland Cement Mortar and Concrete": *Transactions,* American Society of Civil Engineers, Vol. LXXIV, 1911, p. 255.

George W. Rafter.—"Consistency and Proportions": Tests of Metals, U. S. A., 1898.

M. Rudeloff.—*"Versuche mit Eisenbeton-Säulen, Beton und Eisen,* March 9, 1911.

F. Schüle.—*"Resultate der Untersuchung von Armierten Beton,"* Zürich, 1906.

Arthur N. Talbot.—"Prisms, Beams, Columns": *Proceedings,* American Society for Testing Materials, Vol. IV, 1904, p. 476; Vol. VII, 1907, p. 382. University of Illinois, *Bulletins, Nos. 1, 4, 8, 10, 12, 14, 20, 22, 28, 29.*

Arthur N. Talbot and Arthur R. Lord.—"Concrete as Reinforcement for Structural Steel Columns": University of Illinois, *Bulletin, No. 56.*

Sanford E. Thompson.—"Permeability and Consistency": *Proceedings,* American Society for Testing Materials, Vol. VI, 1906, p. 358, and Vol. VIII, 1908, p. 500.

Frederick E. Turneaure.—"Beams, Columns": *Proceedings,* American Society for Testing Materials, Vol. IV, 1904, p. 498.

U. S. Geological Survey Tests, under direction of Richard L. Humphrey. Tests of High-pressure Steam on Concrete, and of Damp-proofing and Water-proofing Compounds; Published by Bureau of Standards, Technologic Papers, 3 and 5.

John L. Van Ornum.—"Fatigue in Reinforced Beams": *Transactions,* American Society of Civil Engineers, Vol. LVIII, 1907, p. 294.

Morton O. Withey.—"Beams, Columns": University of Wisconsin, *Bulletins,* Vol. IV, *Nos. 1, 2;* Vol. V, *Nos. 2, 5.*

Ira H. Woolson.—"Effect of Heat": *Proceedings,* American Society for Testing Materials, Vol. VI, 1906, p. 433, and Vol. VII, 1907, p. 404.

Recommendations of British Reinforced Concrete Committee, 1907, 1911.

Regulations of Prussian Government, 1904, 1907.

Rules of French Government, 1907.

Recommendations of Swiss Society of Engineers and Architects, 1909.

Rules of the Austrian Ministry of the Interior, 1908, 1911.

In addition to the authorities above quoted, the Committee desires to acknowledge with thanks the discussions of its progress reports, which have appeared from time to time, and to say that all the points brought out therein have been carefully weighed.

4.—Character of Report Presented.

At the time of the appointment of the Committee, in 1904, there existed a great diversity of opinion in America as to methods of design, safe allowable working stresses, and methods of proportioning, handling, etc. A great deal of experimental work had been done, but there was need of a clearing house through which results could be compared and divergent views harmonized. During the interval between the appointment of the Committee and the preparation of its first Progress Report rapid advance was made in the art of concrete construction aided by the results of the investigations and the experience acquired by constructors. This report, which was submitted in 1909, attempted to embody recommendations for safe methods of construction and design in accordance with the best practice of the day. It would have been impossible for such a report to meet with the approval of all, and the Committee is well satisfied that its most vital recommendations have met with quite general acceptance by the engineers of the country.

Since the appearance of the first Progress Report, many experiments have been conducted by some of the technical institutions and by private and corporate interests, and through these and through longer experience in construction by its members and others, the Committee is now able to make some perfecting modifications of its former report and to add some entirely new material. The time, therefore, seems opportune for presenting this second report, bringing the work up to date.

The Committee would point out that while the report deals with every kind of stress to which concrete is subjected, and includes all ordinary conditions of proportioning and handling, it does not go into all types of construction or all the applications to which concrete and reinforced concrete may be put.

It is not to be assumed that the Committee in presenting this report wishes to imply that further improvements are not possible. A careful reading will disclose many points on which the present deductions are regarded as only tentative; but it has been the aim of the Committee to cover as fully as possible recommendations based on the present state of the art.

This report is what the word implies, and nothing more; it is not a "specification", but may be used as a basis for specifications.

The use of concrete and reinforced concrete involves the exercise of good judgment to a greater degree than for any other building material.

Rules cannot produce or supersede judgment; on the contrary, judgment should control the interpretation and application of rules.

II. Adaptability of Concrete and Reinforced Concrete.

The adaptability of concrete and reinforced concrete for engineering structures, or parts thereof, is now so well established that they may be considered the recognized materials of construction. They have proved satisfactory materials, when properly used, for those purposes for which their qualities make them particularly suitable.

1.—Uses.

Concrete is a material of very low tensile strength, and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly on its durability, its fire-resistive qualities, its strength in compression, its relatively low cost, and its adaptability to placing, especially where space is cramped or limited. Its strength increases generally with age.

Concrete is well adapted for structures in which the principal stresses are compressive, such as foundations, dams, retaining and other walls, tunnels, piers, abutments, short columns, and, in many cases, arches. In the design of massive concrete, the tensile strength of the material in resisting principal stresses must generally be neglected.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where

the main stresses are compressive, but where cross-bending may exist. In structures resisting lateral forces, it possesses advantages over plain concrete in that it may be designed so as to utilize more fully the strength rather than the weight of the material. Metal reinforcement may also be of value in distributing cracks due to shrinkage and temperature changes.

2.—Precautions.

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: defective design, poor material, faulty execution, and premature removal of forms.

The defects in a design may be many and various. The computations and assumptions on which they were based may be faulty and contrary to the established principles of statics and mechanics; the unit stresses used may be excessive, or the details of the design defective.

Articulated concrete structures designed in imitation of steel trusses, may be mentioned as illustrating a questionable use of reinforced concrete, and such structures are not recommended.

Poor material is sometimes used for the concrete, as well as for the reinforcement. The use of poor aggregates, especially sand, which have not been tested, is a common source of defect. Inferior concrete is frequently due also to lack of experience on the part of the contractor and his superintendents, or to the absence of proper supervision.

An unsuitable quality of metal for reinforcement is sometimes prescribed in specifications, for the purpose of reducing the cost. For steel structures, a high grade of material is specified, but the steel used for reinforcing concrete is sometimes made of unsuitable, brittle material.

Faulty execution, careless workmanship, and too early removal of forms may generally be attributed to unintelligent or insufficient supervision.

3.—Responsibility and Supervision.

The design of reinforced concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed

study of such connections, accompanied by computations to prove their strength.

While other engineering structures on the safety of which human lives depend are generally designed by engineers employed by the owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures frequently are designed by contractors or by engineers commercially interested, and the contract let for a lump sum.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Therefore, means for more adequate supervision and inspection should be provided.

The execution of the work should not be separated from the design, as intelligent supervision and successful execution can be expected only when both functions are combined. The engineer who prepares the design and specifications, therefore, should have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete for the purpose of fixing the responsibility and providing for adequate supervision during construction:

(*a*) Before work is commenced, complete plans shall be prepared, accompanied by specifications, stress computations, and descriptions showing the general arrangement and all details. The plans shall show the size, length, dimensions for points of bending, and exact position of all reinforcement, including stirrups, ties, hooping, and splicing. The computations shall give the loads assumed separately, such as dead and live loads, wind, and impact, if any, and the resulting stresses.

(*b*) The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

(*c*) The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

(*d*) The drawings and specifications shall be signed by the engineer and the contractor.

(*e*) Plans and specifications for all public structures should be approved by a legally authorized State or City official, and copies of such plans and specifications placed on file in his office.

(*f*) The approval of plans and specifications by other authorities shall not relieve the engineer or the contractor of responsibility.

(*g*) Inspection during construction shall be made by competent inspectors employed by and under the supervision of the engineer, and shall cover the following:

1. The materials.
2. The correct construction and erection of the forms and the supports.
3. The sizes, shapes, and arrangement of the reinforcement.
4. The proportioning, mixing, and placing of the concrete.
5. The strength of the concrete, by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Prevention of injury to any part of the structure by and after the removal of the forms.
8. Comparison of dimensions of all parts of the finished structure with the plans.

(*h*) Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that one and three-quarters times the calculated working stresses in critical parts are reached, and such loads shall cause no injurious permanent deformations. Load tests shall not be made until after 60 days of hardening.

4.—Destructive Agencies.

(*a*) *Corrosion of Metal Reinforcement.*—Tests and experience indicate that steel sufficiently embedded in good concrete is well protected against corrosion, no matter whether located above or below water level. It is recommended that such protection be not less than 1 in. in thickness. If the concrete is porous, so as to be readily permeable by water, as when the concrete is laid with a very dry consistency, the metal may corrode on account of the presence of moisture and air.

(*b*) *Electrolysis.*—The most recent experimental data available on this subject seem to show that while reinforced concrete structures may, under certain conditions, be injured by the flow of electric current in either direction between the reinforcing material and the concrete, such injury is generally to be expected only where voltages are considerably higher than those which usually occur in concrete structures in practice. If the iron be positive, trouble may manifest itself by corrosion of the iron accompanied by cracking of the concrete, and, if the iron be negative, there may be a softening of the concrete near the surface of the iron, resulting in a destruction of the bond. The former, or anode effect, decreases much more rapidly than the voltage, and almost if not quite disappears at voltages that are most likely to be encountered in practice. The cathode effect, on the other hand, takes place even on very low voltages, and is therefore more important from a practical standpoint than that of the anode.

Structures containing salt or calcium chloride, even in very small

quantities, are very much more susceptible to the effects of electric currents than normal concrete, both the anode and cathode effects progressing much more rapidly in the presence of chlorine.

There is great weight of evidence to show that normal reinforced concrete structures free from salt are in very little danger under most practical conditions, while non-reinforced concrete structures are practically immune from electrolysis troubles.

The results of experiments now in progress may yield more conclusive information on this subject.

(*c*) *Sea Water.*—The data available concerning the effect of sea water on concrete or reinforced concrete are limited and inconclusive. Sea walls out of the range of frost action have been standing for many years without apparent injury. In many harbors where the water is brackish, through rivers discharging into them, serious disintegration has taken place. This has occurred chiefly between low and high tide levels, and is due, evidently, in part to frost. Chemical action also appears to be indicated by the softening of the mortar. To effect the best resistance to sea water, the concrete must be proportioned, mixed, and placed so as to prevent the penetration of sea water into the mass or through the joints. The cement should be of such chemical composition as will best resist the action of sea water; the aggregates should be carefully selected, graded, and proportioned with the cement so as to secure the maximum possible density; the concrete should be thoroughly mixed; the joints between old and new work should be made water-tight; and the concrete should be kept from exposure to sea water until it is thoroughly hard and impervious.

(*d*) *Acids.*—Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials. A substance like manure is injurious to green concrete, but after the concrete has hardened thoroughly it resists the action of such acid satisfactorily.

(*e*) *Oils.*—When concrete is properly made and the surface is carefully finished and hardened, it resists the action of such mineral oils as petroleum and ordinary engine oils. Oils which contain fatty acids produce injurious effects, forming compounds with the lime which result in a disintegration of the concrete in contact with them.

(*f*) *Alkalies.*—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron, and other materials are injured under certain conditions. It would seem that at the level of the ground-water, in an extremely dry atmosphere, such structures are disintegrated, through the rapid crystallization of the alkaline salts, resulting from the alternate wetting and drying of the surface. Such destructive action can be prevented by the use of a protective coating, and is minimized by securing a dense concrete.

III. Materials.

A knowledge of the properties of the materials entering into concrete and reinforced concrete is the first essential. The importance of the quality of the materials used cannot be overestimated, and not only the cement but also the aggregates should be subject to such definite requirements and tests as will insure concrete of the desired quality.

1.—Cement.

There are available for construction purposes: Portland, Natural, and Puzzolan or Slag cements. Only Portland cement is suitable for reinforced concrete.

(*a*) *Portland Cement.*—This is the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials. It has a definite chemical composition varying within comparatively narrow limits.

Portland cement should be used in reinforced concrete construction and any construction that will be subject to shocks or vibrations or stresses other than direct compression.

(*b*) *Natural Cement.*—This is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. Although the limestone must have a certain composition, this composition may vary within much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly, nor is it as uniform in composition, as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of natural cement and a leaner mixture of Portland cement that will develop the same strength.

(*c*) *Puzzolan or Slag Cement.*—This is the finely pulverized product resulting from grinding a mechanical mixture of granulated basic blast furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or natural cement, is not used extensively, and never in important work; it should be used only for foundation work underground where it is not exposed to air or running water.

(*d*) *Specifications.*—The cement should meet the requirements of the Standard Methods of Testing and Specifications for Cement (see Appendix, p. 158), or as may be hereafter amended, the result of the joint labors of Special Committees of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering Association, and others.

2.—Aggregates.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density* or a minimum percentage of voids.

(*a*) *Fine Aggregate.*—This should consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, and passing when dry a screen having holes ¼ in. in diameter; it is preferable that it be of silicious material, and should be clean, coarse, free from dust, soft particles, vegetable loam, or other deleterious matter; and not more than 6% should pass a sieve having 100 meshes per lin. in. Fine aggregates should always be tested.

Fine aggregate should be of such quality that mortar composed of one part Portland cement and three parts fine aggregate by weight, when made into briquettes, will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.† If the aggregate be of poorer quality, the proportion of cement in the mortar should be increased to secure the desired strength.

If the strength developed by the aggregate in the 1:3 mortar is less than 70% of the strength of the Ottawa sand mortar, the material should be rejected. To avoid the removal of any coating on the grains, which may affect the strength, bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined on a separate sample for correcting weight. From 10 to 40% more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

(*b*) *Coarse Aggregate.*—This should consist of crushed stone or gravel which is retained on a screen having holes ¼ in. in diameter, and graded from the smallest to the largest particles; it should be clean, hard, durable, and free from all deleterious matter. Aggregates containing dust, and soft, flat, or elongated particles, should be excluded from important structures.

The maximum size of the coarse aggregate is governed by the character of the construction.

For reinforced concrete and for small masses of unreinforced concrete, the aggregate must be small enough to produce with the mortar a homogeneous concrete of viscous consistency which will pass readily

*A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

†A natural sand obtained at Ottawa, Ill., passing a screen having 20 meshes and retained on a screen having 30 meshes per lin. in.; prepared and furnished by the Ottawa Silica Company, for 2 cents per lb., f. o. b. cars, Ottawa, Ill., under the direction of the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers.

between and easily surround the reinforcement and fill all parts of the forms.

For concrete in large masses, the size of the coarse aggregate may be increased, as a large aggregate produces a stronger concrete than a fine one, although it should be noted that the danger of separation from the mortar becomes greater as the size of the coarse aggregate increases.

Cinder concrete should not be used for reinforced concrete structures. It may be allowable in mass for very light loads or for fire protection purposes. The cinders used should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3.—Water.

The water used in mixing concrete should be free from oil, acid, alkalies, or organic matter.

4.—Metal Reinforcement.

The Committee recommends, as a suitable material for reinforcement, steel filling the requirements for structural steel reinforcement of the specifications adopted by the American Railway Engineering Association (Appendix, p. 161).

Where little bending or shaping is required, and also for reinforcement for shrinkage and temperature stresses, material filling the requirements of the specifications adopted by the American Railway Engineering Association for high-carbon steel (Appendix, p. 161) may be used, adopting the same unit stress as hereinafter recommended for structural grade material.

For the reinforcement of slabs, small beams, or minor details, or for reinforcement for shrinkage and temperature stresses, wire drawn from bars of the grade of rivet steel may be used, with the unit stresses hereinafter recommended.

The reinforcement should be free from excessive rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

IV. Preparing and Placing Mortar and Concrete.

1.—Proportions.

The materials to be used in concrete should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density.

(*a*) *Unit of Measure.*—The unit of measure should be the cubic foot. A bag of cement, containing 94 lb. net, should be considered the equivalent of 1 cu. ft.

The measurement of the fine and coarse aggregates should be by loose volume.

(*b*) *Relation of Fine and Coarse Aggregates.*—The fine and coarse aggregate should be used in such relative proportions as will insure maximum density. In unimportant work it is sufficient to do this by individual judgment, using correspondingly higher proportions of cement; for important work these proportions should be carefully determined by density experiments, and the sizing of the fine and coarse aggregates should be uniformly maintained or the proportions changed to meet the varying sizes.

(*c*) *Relation of Cement and Aggregates.*—For reinforced concrete construction, one part of cement to a total of six parts of fine and coarse aggregates measured separately should generally be used. For columns, richer mixtures are generally preferable, and in massive masonry or rubble concrete, a mixture of 1:9 or even 1:12 may be used.

These proportions should be determined by the strength or the wearing qualities required in the construction at the critical period of its use. Experienced judgment based on individual observation and tests of similar conditions in similar localities is an excellent guide as to the proper proportions for any particular case.

For all important construction, advance tests should be made of concrete, of the materials, proportions, and consistency to be used in the work. These tests should be made under laboratory conditions to obtain uniformity in mixing, proportioning, and storage, and in case the results do not conform to the requirements of the work, aggregates of a better quality should be chosen, or richer proportions used to obtain the desired results.

2.—Mixing.

The ingredients of concrete should be thoroughly mixed, and the mixing should continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous. As the maximum density and greatest strength of a given mixture depend largely on thorough and complete mixing, it is essential that the work of mixing should receive special attention and care.

Inasmuch as it is difficult to determine, by visual inspection, whether the concrete is uniformly mixed, especially where limestone or aggregates having the color of cement are used, it is essential that the mixing should occupy a definite period of time. The minimum time will depend on whether the mixing is done by machine or hand.

(*a*) *Measuring Ingredients.*—Methods of measurement of the proportions of the various ingredients should be used which will secure separate and uniform measurements of cement, fine aggregate, coarse aggregate, and water, at all times.

(*b*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the

materials throughout the mass should be used, as a more uniform consistency can be thus obtained. The mixing should continue for a minimum time of at least 1 min. after all the ingredients are assembled in the mixer.

(c) *Hand Mixing.*—When it is necessary to mix by hand, the mixing should be on a water-tight platform, and especial precautions should be taken to turn all the ingredients together at least six times and until they are homogeneous in appearance and color.

(d) *Consistency.*—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement when used, and which, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

(e) *Retempering.*—Mortar or concrete should not be remixed with water after it has partly set.

3.—Placing Concrete.

(a) *Methods.*—Concrete, after the completion of the mixing, should be handled rapidly, and in as small masses as is practicable, from the place of mixing to the place of final deposit, and under no circumstances should concrete be used that has partly set. A slow-setting cement should be used when a long time is likely to occur between mixing and placing.

Concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper places by gravity, and the surplus water has been forced to the surface. Special care should be exercised to prevent the formation of laitance, which hardens very slowly and forms a poor surface on which to deposit fresh concrete. All laitance should be removed.

Before depositing concrete, the reinforcement should be carefully placed in accordance with the plans, and adequate means provided to hold it in its proper position until the concrete has been deposited and compacted; care should be taken to see that the forms are substantial and thoroughly wetted (except in freezing weather) or oiled, and that the space to be occupied by the concrete is free from débris. When the placing of concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance, thoroughly wetted, and then slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The faces of concrete exposed to premature drying should be kept wet for a period of at least 7 days.

(*b*) *Freezing Weather.*—Concrete should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials covered with ice crystals or containing frost, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

As the coarse aggregate forms the greater portion of the concrete, it is particularly important that this material be heated to well above the freezing point.

(*c*) *Rubble Concrete.*—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced by the use of clean stones thoroughly embedded in the concrete as near together as is possible and still entirely surrounded by concrete.

(*d*) *Under Water.*—In placing concrete under water, it is essential to maintain still water at the place of deposit. The use of tremies, properly designed and operated, is a satisfactory method of placing concrete through water. The concrete should be mixed very wet (more so than is ordinarily permissible) so that it will flow readily through the tremie and into the place with practically a level surface.

The coarse aggregate should be smaller than ordinarily used, and never more than 1 in. in diameter. The use of gravel facilitates mixing and assists the flow of concrete through the tremie. The mouth of the tremie should be buried in the concrete so far that it is at all times entirely sealed and the surrounding water prevented from forcing itself into the tremie; the concrete will then discharge without coming in contact with the water. The tremie should be suspended so that it can be lowered quickly when it is necessary either to choke off or prevent too rapid flow; the lateral flow should preferably be not more than 15 ft.

The flow should be continuous, in order to produce a monolithic mass and prevent the formation of laitance in the interior.

In large structures it may be necessary to divide the mass of concrete into several small compartments or units, filling one at a time. With proper care, it is possible in this manner to obtain as good results under water as in the air.

V. Forms.

Forms should be substantial and unyielding, so that the concrete shall conform to the designed dimensions and contours; and they should be tight in order to prevent the leakage of mortar.

The time for removal of forms is one of the most important steps in the erection of a structure of concrete or reinforced concrete. Care should be taken to inspect the concrete and ascertain its hardness before removing the forms.

So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some competent

and responsible person, especially where the atmospheric conditions are unfavorable.

It may be stated in a general way that forms should remain in place longer for reinforced concrete than for plain or massive concrete, and that forms for floors, beams, and similar horizontal structures should remain in place much longer than for vertical walls.

When the concrete gives a distinctive ring under the blow of a hammer, it is generally an indication that it has hardened sufficiently to permit the removal of the forms with safety. If, however, the temperature is such that there is any possibility that the concrete is frozen, this test is not a safe reliance, as frozen concrete may appear to be very hard.

VI. Details of Construction.

1.—Joints.

(*a*) *Concrete.*—For concrete construction it is desirable to cast the entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joints in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this point, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis of the column, and in girders, beams, and floor slabs perpendicular to the plane of their surfaces.

Girders should never be constructed over freshly formed columns without permitting a period of at least 2 hours to elapse, thus providing for settlement or shrinkage in the columns.

Shrinkage and contraction joints may be necessary in concrete subject to great fluctuations in temperature. The frequency of these joints will depend, first, on the range of temperature to which the concrete will be subjected, and second, on the quantity and position of the reinforcement. These joints should be determined, and provided for in the design. In massive work, such as retaining walls, abutments, etc., built without reinforcement, contraction joints should be provided at intervals of from 25 to 50 ft. and with reinforcement from 50 to 80 ft. (the smaller the height and thickness, the closer the spacing) throughout the length of the structure. To provide against the structure being thrown out of line by unequal settlement, each section of the wall should be tongued and grooved into the adjoining section. A groove should be formed in the surface of the concrete at vertical joints in walls or abutments.

Shrinkage and contraction joints should be lubricated by either an application of petroleum residuum oil or a similar material, so as to permit a free movement at the joint when the concrete expands or contracts.

The insertion of a sheet of copper or zinc, or even tarred paper, will be found advantageous in securing expansion and contraction at the joint.

(*b*) *Reinforcement.*—Wherever it is necessary to splice tension reinforcement, the length of lap should be determined on the basis of the safe bond stress, the stress in the bar, and the shearing resistance of the concrete at the point of splice; or a connection should be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress should be avoided. In columns, bars more than $\frac{3}{4}$ in. in diameter, not subject to tension, should be properly squared and butted in a suitable sleeve; smaller bars may be treated as indicated for tension reinforcement, or the stress may be cared for by embedment in large masses of concrete. At foundations, bearing plates should be provided for supporting the bars, or the bars may be carried into the footing a sufficient distance to transmit the stress of the steel to the concrete by means of the bearing and bond resistance; in no case shall the ends of the bars be permitted merely to rest on concrete.

2.—Shrinkage and Temperature Changes.

Shrinkage of concrete, due to hardening and contraction from temperature changes, causes cracks the size of which depends on the extent of the mass. The resulting stresses are important in monolithic construction, and should be considered carefully by the designer; they cannot be counteracted successfully, but the effects can be minimized.

Large cracks, produced by quick hardening or wide ranges of temperature, can be broken up to some extent into small cracks by placing reinforcement in the concrete; in long continuous lengths of concrete, it is better to provide shrinkage joints at points in the structure where they will do little or no harm. Reinforcement is of assistance, and permits longer distances between shrinkage joints than when no reinforcement is used.

Small masses or thin bodies of concrete should not be joined to larger or thicker masses without providing for shrinkage at such points. Fillets similar to those used in metal castings, but of larger dimensions, for gradually reducing from the thicker to the thinner body, are of advantage.

Shrinkage cracks are likely to occur at points where fresh concrete is joined to that which is set, and hence, in placing the concrete, construction joints should be made on horizontal and vertical lines, and, if possible, at points where joints would naturally occur in dimension-stone masonry.

3.—Fire-Proofing.

The actual fire tests of concrete and reinforced concrete have been limited, but experience, together with the results of tests thus far made, indicates that concrete, on account of its low rate of heat conductivity and the fact that it is incombustible, may be used safely for fire-proofing purposes.

The dehydration of concrete probably begins at about 500° Fahr., and is completed at about 900° Fahr., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is actually affected by fire remains in position and affords protection to the concrete beneath it.

The thickness of the protective coating required depends on the probable duration of a fire which is likely to occur in the structure, and should be based on the rate of heat conductivity. The question of the conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions, it is recommended that the metal in girders and columns be protected by a minimum of 2 in. of concrete; that the metal in beams be protected by a minimum of 1½ in. of concrete; and that the metal in floor slabs be protected by a minimum of 1 in. of concrete.

It is recommended that, in monolithic concrete columns, the concrete to a depth of 1½ in. be considered as protective covering, and not included in the effective section.

It is recommended that the corners of columns, girders, and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

4.—Water-Proofing.

Many expedients have been used to render concrete impervious to water under normal conditions, and also under pressure conditions that exist in reservoirs, dams, and conduits of various kinds. Experience shows, however, that where mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a rather wet consistency, the resulting mortar or concrete is impervious under moderate pressure.

A concrete of dry consistency is more or less pervious to water, and compounds of various kinds have been mixed with the concrete, or applied as a wash to the surface for the purpose of making it water-tight. Many of these compounds are of but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls, and reservoirs, provided the concrete itself is impervious, cracks may be so reduced by

horizontal and vertical reinforcement properly proportioned and located, that they are too minute to permit leakage, or are soon closed by infiltration of silt.

Coal-tar preparations, applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

For retaining and similar walls in direct contact with the earth, the application of one or two coatings of hot coal-tar pitch to the thoroughly dried surface of concrete is an efficient method of preventing the penetration of moisture from the earth.

5.—Surface Finish.

Concrete is a material of an individual type, and should not be used in imitation of other structural materials. One of the important problems connected with its use is the character of the finish of exposed surfaces. The finish of the surface should be determined before the concrete is placed, and the work should be conducted so as to make possible the finish desired. For many forms of construction the natural surface of the concrete is unobjectionable, but frequently the marks of the boards and the flat dead surface are displeasing, making some special treatment desirable. A treatment of the surface, either by scrubbing it while green or by tooling it after it is hard, which removes the film of mortar and brings the aggregates of the concrete into relief is frequently used to remove the form markings, break the monotonous appearance of the surface, and make it more pleasing. The plastering of surfaces, as ordinarily applied, should be avoided, for, even if carefully done, it is likely to peel off under the action of frost or temperature changes.

VII. Design.

1.—Massive Concrete.

In the design of massive or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be proportioned so as to avoid tensile stresses, except in slight amounts to resist indirect stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but, in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete, therefore, will often be suitable for massive concrete structures.

It is desirable, generally, to provide joints at intervals, to localize the effect of contraction.

Massive concrete is suitable for dams, retaining walls, and piers and short columns in which the ratio of length to least width is relatively small. Under ordinary conditions, this ratio should not exceed six. It is also suitable for arches of moderate span, where the conditions as to foundations are favorable.

2.—Reinforced Concrete.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design, therefore, will relate mainly to the analysis of beams and columns.

3.—General Assumptions.

(*a*) *Loads.*—The loads or forces to be resisted consist of:

1. *The Dead Load.*—This includes the weight of the structure and fixed loads and forces.
2. *The Live Load.*—This consists of the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Any allowance for the dynamic effect is preferably taken into account by adding the desired amount to the live load or to the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses thus determined.

   In the case of high buildings, the live load on columns may be reduced in accordance with the usual practice.

(*b*) *Lengths of Beams and Columns.*—The span length for beams and slabs shall be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

(*c*) *Internal Stresses.*—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations will be made with reference to working stresses and safe loads, rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams, therefore, is rectilinear.

4. In calculating the moment of resistance of beams, the tensile stresses in the concrete are neglected.
5. Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses, the two materials, therefore, are stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15, except as modified in Chapter VIII, Section 8.
7. Initial stress in the reinforcement, due to contraction or expansion in the concrete, is neglected.

It is recognized that some of the assumptions given herein are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

The deflection of beams is affected by the tensile strength developed throughout the length of the beam. For calculations of deflections, a value of 8 for the ratio of the moduli will give results corresponding approximately with the actual conditions.

4.—T-Beams.

In beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

(*a*) It shall not exceed one-fourth of the span length of the beam;

(*b*) Its overhanging width on either side of the web, shall not exceed four times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support.

Beams in which the tee form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form, the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in design.

5.—Floor Slabs.

Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds one and five-tenth times its

width, the entire load should be carried by transverse reinforcement. Square slabs may well be reinforced in both directions.*

The continuous flat slab with multiple-way reinforcement is a type of construction used quite extensively, and has recognized advantages for special conditions, as in the case of warehouses with large, open, floor space. At present, a considerable difference of opinion exists among engineers as to the formulas and constants which should be used, but experience and tests are accumulating data which it is hoped will in the near future permit the formulation of the principles of design for this form of construction.

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and its distribution will depend on the relative stiffness of the slab and the supporting beam. The distribution under ordinary conditions of construction may be expected to be that in which the load on the beam varies in accordance with the ordinates of a parabola having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained, and the moments in the beam calculated accordingly.

6.—Continuous Beams and Slabs.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment, and the stresses in concrete recommended in Chapter VIII, Section 4, should not be exceeded. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(*a*) That for floor slabs, the bending moments at center and at support be taken at $\frac{wl^2}{12}$ for both dead and live loads, where w represents the load per linear foot and l the span length.

* The exact distribution of load on square and rectangular slabs, supported on four sides and reinforced in both directions, cannot readily be determined. The following method of calculation is recognized as faulty, but it is offered as a tentative method which will give results on the safe side. The distribution of load is first to be determined by the formula:

$$r = \frac{l^4}{l^4 + b^4}$$

in which r = proportion of load carried by the transverse reinforcement, l = length, and b = breadth of slab. For various ratios of $\frac{l}{b}$ the values of r are as follows:

$\frac{l}{b}$	r
1	0.50
1.1	0.59
1.2	0.67
1.3	0.75
1.4	0.80
1.5	0.83

Using the values above specified, each set of reinforcement is to be calculated in the same manner as slabs having supports on two sides only, but the total amount of reinforcement thus determined may be reduced 25%, by gradually increasing the rod spacing from the third point to the edge of the slab.

(*b*) That for beams, the bending moment at center and at support for interior spans be taken at $\frac{wl^2}{12}$, and for end spans it be taken at $\frac{wl^2}{10}$ for center and adjoining support, for both dead and live loads.

(*c*) In the case of beams and slabs continuous for two spans only, the bending moment at the central support should be taken as $\frac{wl^2}{8}$ and near the middle of the span as $\frac{wl^2}{10}$.

(*d*) At the ends of continuous beams, the amount of negative moment which will be developed will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. There will usually be some restraint, and there is likely to be considerable. Provision should be made for the negative bending moment, but, as its amount will depend on the form of construction, the coefficient cannot be specified here, and must be left to the judgment of the designer.

For spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by *a* or *b*, the negative moment at the support should not be taken as less than the values there given.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress, in accordance with the provisions of Chapter VII, Section 3, *c*-6. In the case of cantilever and continuous beams, tensile and compressive reinforcement over supports must extend sufficiently beyond the support and beyond the point of inflection to develop the requisite bond strength.

7.—Bond Strength, and Spacing of Reinforcement.

Adequate bond strength should be provided. The formula hereinafter given for bond stresses in beams is for straight longitudinal bars. In beams in which a portion of the reinforcement is bent up near the end, the bond stress at places in both the straight bars and the bent bars will be considerably greater than for all the bars straight, and the stress at some point may be several times as much as that found by considering the stress to be uniformly distributed along the bar. In restrained and cantilever beams, full tensile stress exists in the reinforcing bars at the point of support, and the bars must be anchored in the support sufficiently to develop this stress.

In case of anchorage of bars, an additional length of bar must be provided beyond that found on the assumption of uniform bond stress, for the reason that, before the bond resistance at the end of the bar can be developed, the bar may have begun to slip at another point, and "running" resistance is less than the resistance before slip begins.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength; but it should be recognized that, even with a deformed bar, initial slip occurs at early loads, and that the ultimate loads obtained in the usual tests for bond resistance may be misleading. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard, such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than hooks consisting of turns through 180 degrees.

The lateral spacing of parallel bars should not be less than three diameters, from center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than 1 in. The use of more than two layers is to be discouraged, unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or bent down.

8.—Diagonal Tension and Shear.

When a reinforced concrete beam is subjected to flexural action, diagonal tensile stresses are set up. If, in a beam not having web reinforcement, these stresses exceed the tensile strength of the concrete, failure of the beam will ensue. When web reinforcement, made up of stirrups, or of diagonal bars secured to the longitudinal reinforcement, or of longitudinal reinforcing bars bent up at several points, is used, new conditions prevail; but even in this case, at the beginning of loading, the diagonal tension developed is taken principally by the concrete, the deformations which are developed in the concrete permitting but little stress to be taken by the web reinforcement. When the resistance of the concrete to the diagonal tension is overcome at any point in the depth of the beam, greater stress is at once set up in the web reinforcement.

For homogeneous beams, the analytical treatment of diagonal tension is not very complex—the diagonal tensile stress is a function of the horizontal and vertical shearing stresses and of the horizontal tensile stress at the point considered, and as the intensity of these three stresses varies from the neutral axis to the remotest fiber, the intensity of the diagonal tension will be different at different points in the section, and will change with different proportionate dimensions of length to depth of beam. For the composite structure of reinforced concrete beams, an

analysis of the web stresses, and particularly of the diagonal tensile stresses is very complex; and when the variations due to a change from no horizontal tensile stress in the concrete at the remotest fiber to the presence of horizontal tensile stress at some point below the neutral axis are considered, the problem becomes more complex and indefinite. Under these circumstances, in designing, recourse is had to the use of the calculated vertical shearing stress as a means of comparing or measuring the diagonal tensile stresses developed, it being understood that the vertical shearing stress is not the numerical equivalent of the diagonal tensile stress, and even that there is not a constant ratio between them. It is here recommended that the maximum vertical shearing stress in a section be used as the means of comparison of the resistance to diagonal tensile stress developed in the concrete in beams not having web reinforcement.

Even after the concrete has reached its limit of resistance to diagonal tension, if the beam has web reinforcement, conditions of beam action will continue to prevail, at least through the compression area, and the web reinforcement will be called on to resist only a part of the web stresses. From experiments with beams, it is concluded that it is safe practice to use only two-thirds of the external vertical shear in making calculations of the stresses that come on stirrups, diagonal web pieces, and bent-up bars, and it is here recommended for calculations in designing that two-thirds of the external vertical shear be taken as producing stresses in web reinforcement.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that vertical members attached to or looped about horizontal members, inclined members secured to horizontal members in such a way as to insure against slip, and the bending of a part of the longitudinal reinforcement at an angle, will increase the strength of a beam against failure by diagonal tension, and that a well-designed and well-distributed web reinforcement may, under the best conditions, increase the total vertical shear carried to a value as much as three times that obtained when the bars are all horizontal and no web reinforcement is used. Where vertical stirrups are used without being secured to the longitudinal reinforcement, the force transmitted between longitudinal bar and stirrup must not be greater than can be taken through the concrete, and care must be taken to provide for the larger bond stress developed in the longitudinal bars with this construction than exists in the absence of stirrups. Sufficient bond resistance between the concrete and the stirrups or diagonals must be provided. Where the longitudinal bars are bent up, the points of bending of the several bars should be distributed along a portion of the length of the beam in such a way as to give efficient

web reinforcement over the portion of the length of the beam in which it is needed. The higher resistance to diagonal tension failures given by unit frames having the stirrups and bent-up bars securely connected together both longitudinally and laterally is worthy of recognition. It is necessary that a limit be placed on the amount of shear which may be allowed in a beam; for when web reinforcement sufficiently efficient to give very high web resistance is used, at the higher stresses the concrete in the beam becomes checked and cracked in such a way as to endanger its durability as well as its strength.

The section to be taken as the critical section in the calculation of shearing stresses will generally be the one having the maximum vertical shear, though experiments show that the section at which diagonal tension failures occur is not just at a support, even though the shear at the latter point be much greater.

The longitudinal spacing of stirrups or diagonal members, or the distribution of the points of bending of adjacent bent-up bars, should not exceed three-fourths the depth of the beam.

It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement.

It should be noted that it is on the tension side of a beam that diagonal tension develops in a critical way, and that the proper connection must always be made between stirrups or other web reinforcement and the longitudinal tension reinforcement, whether the latter is on the lower side of the beam or on its upper side. Where negative moment exists, as is the case near the supports in a continuous beam, web reinforcement, to be effective, must be looped over, or wrapped around, or be connected with, the longitudinal tension reinforcing bars at the top of the beam, in the same way as is necessary at the bottom of the beam at sections where the bending moment is positive and the tension reinforcing bars are at the bottom of the beam.

Inasmuch as the smaller the longitudinal deformations in the horizontal reinforcement are, the less the tendency for the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by arranging and proportioning the horizontal reinforcement so that the unit stresses at points of large shear shall be relatively low.

Where pure shearing stress occurs, or shearing stress combined with but a small amount of tensile stress in the concrete, as when a concentrated load rests on a slab, or other forms of punching shear are produced, or in the case of compression pieces, the element of tension will not need consideration, and the permissible limit of the shearing stress will be higher than the allowable limit when this stress is used as a means of comparing diagonal tensile stress. The working values recommended are given in Chapter VIII, Working Stresses.

9.—Columns.

By columns are meant compression members of which the ratio of unsupported length to least width exceeds about six, and which are provided with reinforcement of one of the forms hereafter described.

It is recommended that the ratio of unsupported length of column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering, as defined in Chapter VI, Section 3; or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by longitudinal bars, by bands, hoops, or spirals, together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of closely spaced hooping is greatly to increase the "toughness" of the column and its ultimate strength, but hooping has little effect on its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

Composite columns of structural steel and concrete in which the steel forms a column by itself, should be designed with caution. To classify this type as a concrete column reinforced with structural steel is hardly permissible, as the steel will generally take the greater part of the load. When this type of column is used, the concrete should not be relied on to tie the steel units together or to transmit stresses from one unit to another. The units should be adequately tied together by tie-plates or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. The concrete may exert a beneficial effect in restraining the steel from lateral deflection, and also in increasing the carrying capacity of the column. The proportion of load to be carried by the concrete will depend on the form of the column and the method of construction. Generally, for high percentages of steel, the concrete will develop relatively low unit stresses, and caution should be used in placing dependence on the concrete.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

(*a*) Columns with longitudinal reinforcement only, to the extent of not less than 1% and not more than 4%: the unit stress recommended for axial compression in Chapter VIII, Section 3.

(*b*) Columns with reinforcement of bands, hoops, or spirals, as hereinafter specified: stresses 20% higher than given for *a*, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

(*c*) Columns reinforced with not less than 1% and not more than 4% of longitudinal bars, and with bands, hoops, or spirals, as hereinafter specified: stresses 45% higher than given for *a*, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

The foregoing recommendations are based on the following conditions:

In all cases, longitudinal reinforcement is assumed to carry its proportion of stress, in accordance with Section 3. The hoops or bands are not to be counted on directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where hooping is used, the total amount of such reinforcement shall be not less than 1% of the volume of the column enclosed. The clear spacing of such hooping shall be not greater than one-sixth of the diameter of the enclosed column, and preferably not greater than one-tenth, and in no case more than 2½ in. Hooping is to be circular, and the ends of bands must be united in such a way as to develop their full strength. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered. The strength of hooped columns depends very much on the ratio of length to diameter of hooped core, and the strength due to hooping decreases rapidly as this ratio increases beyond five. The working stresses recommended are for hooped columns with a length of not more than eight diameters of the hooped core.

Bending stresses due to eccentric loads and lateral forces must be provided for by increasing the section until the maximum stress does not exceed the values above specified; and, where tension is possible in the longitudinal bars, adequate connection between the ends of the bars must be provided to take this tension.

10.—Reinforcing for Shrinkage and Temperature Stresses.

When areas of concrete too large to expand and contract freely as a whole are exposed to atmospheric conditions, the changes of form due to shrinkage (resulting from hardening) and to action of temperature are such that cracks may occur in the mass, unless precautions are taken to distribute the stresses so as to prevent the cracks alto-

gether, or to render them very small. The distance apart of the cracks, and consequently their size, will be directly proportional to the diameter of the reinforcement and to the tensile strength of the concrete, and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement (in amount generally not less than one-third of 1%) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed. The allowable size and spacing of cracks depends on various considerations, such as the necessity for water-tightness, the importance of appearance of the surface, and the atmospheric changes.

VIII. Working Stresses.

1.—General Assumptions.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may have approximately the same degree of safety.

The following recommendations as to allowable stresses are given in the form of percentages of the ultimate strength of the particular concrete which is to be used; this ultimate strength is to be that developed in cylinders 8 in. in diameter and 16 in. long of the consistency described in Chapter IV, Section 2 (*a*), made and stored under laboratory conditions, at an age of 28 days. In the absence of definite knowledge, in advance of construction, as to just what strength may be expected, the Committee submits the following values as those which should be obtained with materials and workmanship in accordance with the recommendations of this report.

Although occasional tests may show higher results than those here given, the Committee recommends that these values should be the maximum used in design.

Table of Strengths of Different Mixtures of Concrete.

(In Pounds per Square Inch.)

Aggregate.	1:1:2	1:1½:3	1:2:4	1:2½:5	1:3:6
Granite, trap rock	3 300	2 800	2 200	1 800	1 400
Gravel, hard limestone and hard sandstone.	3 000	2 500	2 000	1 600	1 300
Soft limestone and sandstone	2 200	1 800	1 500	1 200	1 000
Cinders	800	700	600	500	400

Note.—For variations in the moduli of elasticity see Chapter VIII, Section 8.

2.—Bearing.

When compression is applied to a surface of concrete of at least twice the loaded area, a stress of 32.5% of the compressive strength may be allowed.

3.—Axial Compression.

For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.5% of the compressive strength may be allowed.

For other forms of columns the stresses obtained from the ratios given in Chapter VII, Section 9, may govern.

4.—Compression in Extreme Fiber.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5% of the compressive strength. Adjacent to the support of continuous beams stresses 15% higher may be used.

5.—Shear and Diagonal Tension.

In calculations on beams in which the maximum shearing stress in a section is used as the means of measuring the resistance to diagonal tension stress, the following allowable values for the maximum vertical shearing stress are recommended:

(*a*) For beams with horizontal bars only and without web reinforcement calculated by the method given in the Appendix, Formula (22): 2% of the compressive strength.

(*b*) For beams thoroughly reinforced with web reinforcement: the value of the shearing stress calculated as for *a* (that is, using the total external vertical shear in Formula (22) for shearing unit stress), must not exceed 6% of the compressive strength. The web reinforcement, exclusive of bent-up bars, in this case, shall be proportioned to resist two-thirds of the external vertical shear in the formulas given in the Appendix, Formulas (24) or (25).

(*c*) For beams in which part of the longitudinal reinforcement is used in the form of bent-up bars distributed over a portion of the beam in a way covering the requirements for this type of web reinforcement: the limit of maximum vertical shearing stress (the stress calculated as for *a*), 3% of the compressive strength.

(*d*) Where punching shear occurs, that is, shearing stress uncombined with compression normal to the shearing surface, and with all tension normal to the shearing plane provided for by reinforcement: a shearing stress of 6% of the compressive strength may be allowed.

6.—Bond.

The bond stress between concrete and plain reinforcing bars may be assumed at 4% of the compressive strength, or 2% in the case of drawn wire.

7.—Reinforcement.

The tensile or compressive strength in steel should not exceed 16 000 lb. per sq. in.

In structural steel members, the working stresses adopted by the American Railway Engineering Association are recommended.

8.—Modulus of Elasticity.

The value of the modulus of elasticity of concrete has a wide range, depending on the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in computations for the position of the neutral axis and for the resisting moment of beams and for the compression of concrete in columns it be assumed as:

(*a*) One-fifteenth of that of steel, when the strength of the concrete is taken as 2 200 lb. per. sq. in. or less.

(*b*) One-twelfth of that of steel, when the strength of the concrete is taken as greater than 2 200 lb. per sq. in., or less than 2 900 lb. per sq. in., and

(*c*) One-tenth of that of steel, when the strength of the concrete is taken as greater than 2 900 lb. per sq. in.

Although not rigorously accurate, these assumptions will give safe results. For the deflection of beams which are free to move longitudinally at the supports, in using formulas for deflection which do not take into account the tensile strength developed in the concrete, a modulus one-eighth of that of steel is recommended.

Respectfully submitted,

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IX. Appendix.

1.—Standard Specifications.

(*a*) *Cement.**

GENERAL OBSERVATIONS.

1.—These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2.—The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

SPECIFIC GRAVITY.

3.—Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of .the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

FINENESS.

4.—The sieves should be kept thoroughly dry.

TIME OF SETTING.

5.—Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities, vitally affect the rate of setting.

CONSTANCY OF VOLUME.

6.—The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

7.—In making the pats, the greatest care should be exercised to avoid initial strains due to moulding or to too rapid drying-out during the first 24 hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

8.—The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement, however, may be held for 28 days, and a retest made at the end of that period, using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our

* Adopted August 16th, 1909, by the American Society for Testing Materials.

knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

General Conditions.

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the contractor, and a period of at least 12 days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages, with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 lb. of cement net. Each barrel of Portland cement shall contain 4 bags, and each barrel of natural cement shall contain 3 bags of the above net weight.

8. Cement failing to meet the 7-day requirements may be held awaiting the results of the 28-day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society on January 17th, 1912, with all subsequent amendments thereto.

10. The acceptance or rejection shall be based on the following requirements:

Natural Cement.

11. *Definition.*—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

FINENESS.

12. It shall leave by weight a residue of not more than 10% on the No. 100, and 30% on the No. 200 sieve.

TIME OF SETTING.

13. It shall not develop initial set in less than 10 min., and shall not develop hard set in less than 30 min., or more than 3 hours.

TENSILE STRENGTH.

14. The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Age.	*Neat Cement.* Strength.
24 hours in moist air	75 lb.
7 days (1 day in moist air, 6 days in water)	150 "
28 days (1 day in moist air, 27 days in water)	250 "
One Part Cement, Three Parts Standard Ottawa Sand.	
7 days (1 day in moist air, 6 days in water)	50 lb.
28 days (1 day in moist air, 27 days in water)	125 "

CONSTANCY OF VOLUME.

15. Pats of neat cement about 3 in. in diameter, ½ in. thick at the center, tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another is kept in water maintained as near 70° Fahr. as practicable.

16. These pats are observed at intervals for at least 28 days, and, to pass the tests satisfactorily, should remain firm and hard and show no signs of distortion, checking, cracking, or disintegrating.

PORTLAND CEMENT.

17. *Definition.*—This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3% has been made subsequent to calcination.

SPECIFIC GRAVITY.

18. The specific gravity of cement shall be not less than 3.10. Should the test of cement as received fall below this requirement, a second test may be made on a sample ignited at a low red heat. The loss in weight of the ignited cement shall not exceed 4 per cent.

FINENESS.

19. It shall leave by weight a residue of not more than 8% on the No. 100, and not more than 25% on the No. 200 sieve.

TIME OF SETTING.

20. It shall not develop initial set in less than 30 min.; and must develop hard set in not less than 1 hour, nor more than 10 hours.

TENSILE STRENGTH.

21. The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Age.	*Neat Cement.*	Strength.
24 hours in moist air		175 lb.
7 days (1 day in moist air, 6 days in water)		500 "
28 days (1 day in moist air, 27 days in water)		600 "
One Part Cement, Three Parts Standard Ottawa Sand.		
7 days (1 day in moist air, 6 days in water)		200 lb.
28 days (1 day in moist air, 27 days in water)		275 "

CONSTANCY OF VOLUME.

22. Pats of neat cement about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° Fahr. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for 5 hours.

23. These pats, to pass the requirements satisfactorily, shall remain firm and hard, and show no signs of distortion, checking, cracking, or disintegrating.

SULPHURIC ACID AND MAGNESIA.

24. The cement shall not contain more than 1.75% of anhydrous sulphuric acid (SO_3), nor more than 4% of magnesia (MgO).

(*b*) *Metal Reinforcement.**

6.—Steel shall be made by the open-hearth process. Re-rolled material will not be accepted.

7.—Plates and shapes used for reinforcement shall be of structural steel only. Bars and wire may be of structural steel or high-carbon steel.

8.—The chemical and physical properties shall conform to the following limits:

Elements Considered.	Structural Steel.	High-Carbon Steel.
Phosphorus, maximum, Basic	0.04 per cent.	0.04 per cent.
Phosphorus, maximum, Acid	0.06 " "	0.06 " "
Sulphur, maximum	0.05 " "	0.05 " "
Ultimate tensile strength.	Desired.	Desired.
Pounds, per square inch	60 000	88 000
Elong. min. percentage in 8 in., Fig. 1	1 500 000† / Ult. tensile strength.	1 000 000 / Ult. tensile strength.
Character of fracture	Silky	Silky or finely granular.
Cold bends without fracture	180° flat ‡	180° $d = 4t$ §

* Adopted March 16th, 1910, by the American Railway Engineering Association.

† See Paragraph 15. ‡ See Paragraphs 16 and 17. § "$d = 4t$" signifies "around a pin having a diameter four times the thickness of the specimen."

9.—The yield point for bars and wire, as indicated by the drop of the beam, shall be not less than 60% of the ultimate tensile strength.

10.—If the ultimate strength varies more than 4 000 lb. for structural steel or 6 000 lb. for high-carbon steel, a retest shall be made on the same gauge, which, to be acceptable, shall be within 5 000 lb. for structural steel, or 8 000 lb. for high-carbon steel, of the desired ultimate.

11.—Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for, in which case an excess of 25% above the required limits will be allowed.

12.—*Plates, Shapes, and Bars.*—Specimens for tensile and bending tests for plates and shapes shall be made by cutting, from the finished product, coupons which shall have both faces rolled and both edges

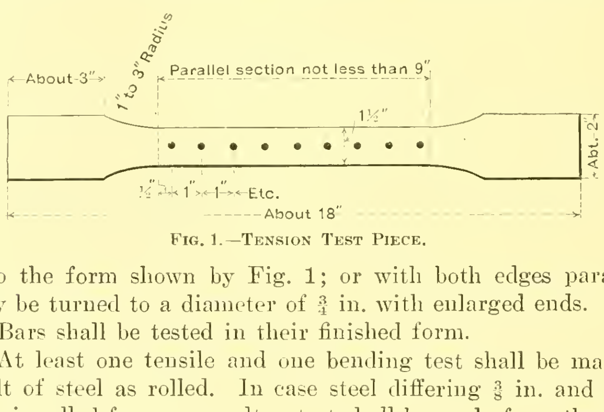


FIG. 1.—TENSION TEST PIECE.

milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾ in. with enlarged ends.

13.—Bars shall be tested in their finished form.

14.—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

15.—For material less than $\frac{5}{16}$ in. and more than ¾ in. in thickness the following modifications will be allowed in the requirements for elongation:

(*a*) For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of 2½ will be allowed from the specified percentage.

(*b*) For each ⅛ in. in thickness above ¾ in., a deduction of 1 will be allowed from the specified percentage.

16.—Bending tests may be made by pressure or by blows. Shapes and bars less than 1 in. thick shall bend as called for in Paragraph 8.

17.—Test specimens 1 in. thick and greater shall bend cold 180° around a pin, the diameter of which, for structural steel, is twice the

thickness of the specimen, and for high-carbon steel, is six times the thickness of the specimen, without fracture on the outside of the bend.

18.—Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and have a smooth, uniform, and workmanlike finish.

19.—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled on it, except that bar steel and other small parts may be bundled, with the above marks on an attached metal tag.

20.—Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected, and shall be replaced by the manufacturer at his own cost.

21.—All reinforcing steel shall be free from excessive rust, loose scale, or other coatings of any character which would reduce or destroy the bond.

2.—Suggested Formulas for Reinforced Concrete Construction.

These formulas are based on the assumptions and principles given in the chapter on design.

(a) *Standard Notation.*

1.—Rectangular Beams.

The following notation is recommended:

f_s = tensile unit stress in steel,
f_c = compressive unit stress in concrete,
E_s = modulus of elasticity of steel,
E_c = modulus of elasticity of concrete,
$n = \frac{E_s}{E_c}$.
M = moment of resistance, or bending moment in general,
A = steel area,
b = breadth of beam,
d = depth of beam to center of steel,
k = ratio of depth of neutral axis to effective depth d,
z = depth of resultant compression below top,
j = ratio of lever arm of resisting couple to depth d,
$jd = d - z$ = arm of resisting couple,
p = steel ratio (not percentage).

2.—T-Beams.

b = width of flange,
b' = width of stem,
t = thickness of flange.

3.—Beams Reinforced for Compression.

A' = area of compressive steel,
p' = steel ratio for compressive steel,
f_s' = compressive unit stress in steel,
C = total compressive stress in concrete,
C' = total compressive stress in steel,
d' = depth to center of compressive steel,
z = depth to resultant of C and C'.

4.—Shear and Bond.

V = total shear,
v = shearing unit stress,
u = bond stress per unit area of bar,
o = circumference or perimeter of bar,
Σ_0 = sum of the perimeters of all bars.

5.—Columns.

A = total net area,
A_s = area of longitudinal steel,
A_c = area of concrete,
P = total safe load.

(*b*) *Formulas.*

1.—Rectangular Beams.

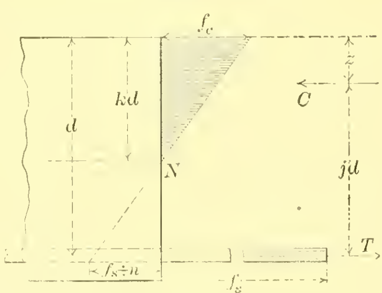


FIG. 2.

Position of neutral axis,

$$k = \sqrt{2\,pn + (pn)^2} - pn \qquad (1)$$

Arm of resisting couple.

$$j = 1 - \frac{1}{3}\,k \qquad (2)$$

(For f_s = 15 000 to 16 000, and f_c = 600 to 650, k may be taken at $\frac{3}{8}$.)

Fiber stresses,

$$f_s = \frac{M}{Ajd} = \frac{M}{pjbd^2} \quad \text{.......................} (3)$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \quad \text{.......................} (4)$$

Steel ratio, for balanced reinforcement,

$$p = \frac{1}{2} \cdot \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)} \quad \text{.......................} (5)$$

2.—T-Beams.

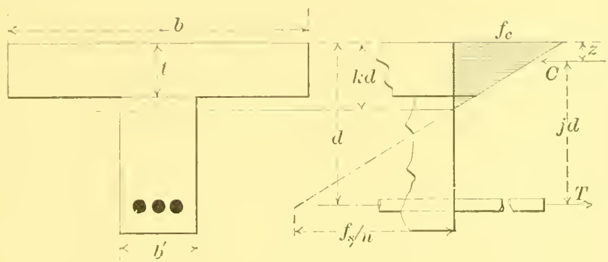


FIG. 3.

Case I. When the neutral axis lies in the flange.
Use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.
The following formulas neglect the compression in the stem:
Position of neutral axis,

$$kd = \frac{2ndA + bt^2}{2nA + 2bt} \quad \text{.......................} (6)$$

Position of resultant compression,

$$z = \frac{3kd - 2t}{2kd - t} \cdot \frac{t}{3} \quad \text{.......................} (7)$$

Arm of resisting couple,

$$jd = d - z \quad \text{.......................} (8)$$

Fiber stresses,

$$f_s = \frac{M}{Ajd} \quad \text{.......................} (9)$$

$$f_c = \frac{Mkd}{bt\left(kd - \frac{1}{2}t\right)jd} = \frac{f_s}{n} \cdot \frac{k}{1-k} \quad \text{.............} (10)$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2\,nd\,A + (b - b')\,t^2}{b'} + \left(\frac{nA + (b - b')\,t}{b'}\right)^2} - \frac{nA + (b - b')\,t}{b'} \quad \text{.......... (11)}$$

Position of resultant compression,

$$z = \frac{\left(kdt^2 - \frac{2}{3}\,t^3\right)\,b + \left[(kd - t)^2\,\left(t + \frac{1}{3}\,(kd - t)\right)\right]b'}{t\,(2\,kd - t)\,b + (kd - t)^2\,b'} \quad \text{.......... (12)}$$

Arm of resisting couple,

$$jd = d - z \quad \text{.......... (13)}$$

Fiber stresses,

$$f_s = \frac{M}{Ajd} \quad \text{.......... (14)}$$

$$f_c = \frac{2\,Mkd}{[(2\,kd - t)\,bt + (kd - t)^2\,b']\,jd} \quad \text{.......... (15)}$$

3.—Beams Reinforced for Compression.

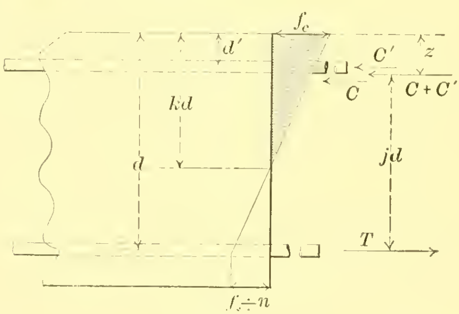


Fig. 4.

Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\,\frac{d'}{d}\right) + n^2\,(p + p')^2} - n\,(p + p') \quad \text{.......... (16)}$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}\,k^3\,d + 2\,p'\,nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2\,p'\,n\left(k - \frac{d'}{d}\right)} \quad \text{.......... (17)}$$

Arm of resisting couple,

$$jd = d - z \quad \text{.......................} (18)$$

Fiber stresses,

$$f_c = \frac{6\,M}{bd^2\left[3\,k - k^2 + \frac{6\,p'\,n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} \quad \text{....} (19)$$

$$f_s = \frac{M}{pjbd^2} = nf_c\,\frac{1 - k}{k} \quad \text{.................} (20)$$

$$f_s' = nf_c\,\frac{k - \frac{d'}{d}}{k} \quad \text{.......................} (21)$$

4.—Shear, Bond, and Web Reinforcement.

In the following formulas, Σ_0 refers only to the bars constituting the tension reinforcement at the section in question, and jd is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v = \frac{V}{bjd} \quad \text{.........................} (22)$$

$$u = \frac{V}{jd\;\Sigma_0} \quad \text{.......................} (23)$$

$\left(\text{For approximate results, } j \text{ may be taken at } \frac{7}{8}.\right)$

The stresses in web reinforcement may be estimated by the following formulas:

Vertical web reinforcement,

$$P = \frac{Vs}{jd} \quad \text{.........................} (24)$$

Web reinforcement inclined at 45° (not bent-up bars),

$$P = 0.7\,\frac{Vs}{jd} \quad \text{.......................} (25)$$

in which P = stress in single reinforcing member, V = amount of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For **T**-beams,

$$v = \frac{V}{b'\,jd} \quad \text{.......................} (26)$$

$$u = \frac{V}{jd\,\Sigma_0} \quad \text{.......................} (27)$$

$\left(\text{For approximate results, } j \text{ may be taken at } \frac{7}{8}.\right)$

5.—Columns.

Total safe load,

$$P = f_c (A_c + n A_s) = f_c A (1 + (n - 1) p) \qquad (28)$$

Unit stresses,

$$f_c = \frac{P}{A (1 + (n - 1) p)} \qquad (29)$$

$$f_s = n f_c \qquad (30)$$

PROGRESS REPORT OF SPECIAL COMMITTEE ON ENGINEERING EDUCATION.

January, 1913.

During the past year the Committee has lost one of its most active members, Past-President B. M. Harrod, who, from the very beginning of the investigation, took an interest in the endeavor to co-ordinate the diverse views entertained by the Profession on almost every phase of Engineering Education.

As the work of the Committee is drawing to a close it is not proposed to fill the vacancy caused by Mr. Harrod's lamented death.

At the time of the last Annual Meeting an understanding had been arrived at with the Carnegie Foundation that they would undertake a comprehensive investigation into the subject of Engineering Education, such an investigation as they had already made into the matter of Medical Education, involving the expenditure of a large sum of money, with satisfactory results.

Owing to the ill health of Dr. Pritchett and his long absence in California, no practical results could be arrived at, but it is now hoped that in a very short time the work will begin.

As the Committee may still be of some service until the new organization is under way, we ask to be continued for another year.

Desmond FitzGerald, *Chairman,*
Onward Bates,
Daniel W. Mead.

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS.

JANUARY 15TH, 1913.

The Special Committee "to consider and report upon the design, ultimate strength, and safe working values of Steel Columns and Struts," presents the following report of progress:

Your Committee submits the proposed programme of 144 column tests as outlined and agreed upon with the representatives of the Bureau of Standards, of the United States Government, S. W. Stratton, Director.

It will be seen that the programme is arranged with the intention, as far as practicable, of eliminating all variables except that of form. The programme is subdivided so as to consider the effect of light and heavy sections. It will be noted, from the accompanying specifications, that the requirements for the material are extremely rigid; in fact, far above the commercial grade. It is expected that the uniform material produced by these specifications will reduce erratic results to a minimum. Your Committee feels that, when the results of these proposed tests of special material have been studied and correlated, the knowledge gained hereby will enable engineers to draw conclusions regarding the action, under load, of columns made of commercial grades of steel.

When its last Progress Report was made, your Committee expected to have the results of this programme for study before now, and to be able to report on the results at this meeting, but it regrets to state that the 2 300 000-lb. testing machine now being installed by the U. S. Government at Washington is not yet ready for operation. The Bureau of Standards writes that the machine will be finished in the near future; that the material for the first half of the programme has been purchased from the American Bridge Company, and that the programme of tests mentioned in this report will be the first to be undertaken.

Your Committee has been unable to avail itself of most of the tests which it has thus far recorded, owing to lack of data regarding the material used. Suggestions have been made regarding the use of discarded columns taken from old structures. If such series of tests are to be of value for comparison with other series, the complete history of the metal should be known.

Last year, your Committee reported that it was in communication with the Committee of the American Railway Engineering Association on Iron and Steel Structures, with a view to the two Committees working in harmony. The sub-committee on Iron and Steel Structures of that Association, W. H. Moore, Chairman, expects to make other series of tests which, correlated with ours, will further the knowledge

of the subject. During the past year the committees of the two Societies have kept in close touch, and their conferences have been of mutual advantage.

Through the loss of its honored member, Mr. Alfred P. Boller, your Committee has been deprived of the wise council and keen judgment of an able and cultured engineer.

Your Committee will be unable to make a final report until the results of the proposed programme are available for study.

For the Committee:

AUSTIN LORD BOWMAN, *Chairman*,
LEWIS D. RIGHTS, *Secretary*.

Committee:

AUSTIN LORD BOWMAN.
EMIL GERBER,
CHAS. F. LOWETH,
RALPH MODJESKI,
FRANK C. OSBORN,
GEO. H. PEGRAM,
GEO. F. SWAIN,
LEWIS D. RIGHTS,
EMIL SWENSSON,
J. R. WORCESTER.

TABLE OF CHEMICAL AND PHYSICAL PROPERTIES FOR SPECIAL SPECIFICATIONS FOR UNIFORM QUALITY STRUCTURAL STEEL FOR COMPARATIVE COLUMN TESTS FOR THE BUREAU OF STANDARDS, U. S. GOVERNMENT.

Material.—Basic open-hearth structural steel.

Chemical.—Uniformity is paramount, and must be aimed for.

Carbon.—The percentage of carbon must not vary more than just to insure the desired physical properties in required sections of material.

Manganese	0.40% min. and 0.50% max.
Phosphorus	0.00% min. and 0.03% max
Sulphur	0.00% min. and 0.04% max.
Silicon	0.00% min. and 0.10% max.
Nickel	0.00% min. and 0.05% max.
Chromium	0.00% min. and 0.05% max.
Copper	0.00% min. and 0.03% max.
All possible others	0.00%

Physical.—The desired physical properties must be aimed for.

Ultimate tensile strength...	Desired, 60 000 lb. per sq. in. (may vary 1 500 lb. either way.)
Elastic limit (yield point)..	Desired, 38 000 lb. per sq. in. (may vary 1 000 lb. either way.)
Elongation in 8 in..........	Desired, 28% (may vary 2% either way.)
Reduction in area..........	Desired, 56% (may vary 4% either way.)
Character of fracture.......	Silky, cup.
Cold bend without fracture..	180°, flat, on itself.

Specifications for Workmanship for Special Test Columns for the Bureau of Standards, U. S. Government.

In general, the American Railway Engineering Association Specifications will govern this work, with the following changes and additions thereto:

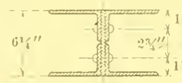
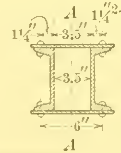
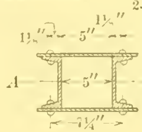
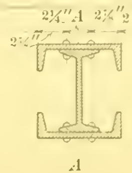
The workmanship required for these columns is to be gild-edged.

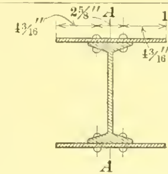
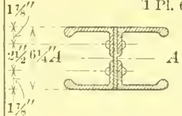
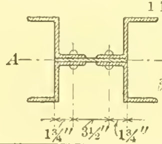
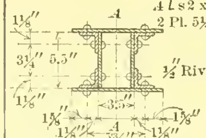
Each piece of rolled material is to be absolutely true to dimensions and made absolutely straight.

The punching must be exact, and holes must be $\frac{3}{16}$ in. smaller than diameter of rivet. The holes must be reamed when pieces are assembled into columns.

The columns will be riveted by stitch-riveting them on alternate sides, in order to prevent warping, and to produce absolutely straight columns. Slight deviations from a straight line of axis, changes in sections or windings, etc., must be carefully removed, so as to produce the most perfect column shafts possible.

Facing of ends to be absolutely at right angles to axis of column, and thus, of course, parallel to each other.

Light Section.	Area in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches	r	$\frac{l}{r}$	Length.
4 Ls 5"×3"×5/16" 1 Pl. 6"×5/16"  3/4" Rivets	9.60 1.875 11.475	2.24	120 85 50	22'4 13/16" 15'10 7/16" 9'1"	4 Ls 5"×3"×5/8" 1 Pl. 6"×5/8" A—Axis—A 3/4" Rivets	18.11 3.75 22.19	2.36	120 85 50	23'7 3/16" 16'8 5/8" 9'10"
2-6" Cs × 10.5-lb. 2 Pl. 8"×1/4" 1/2" Rivets 	6.18 4.00 10.18	2.31	120 85 50	23'1 3/16" 16'4 3/8" 9'7 1/2"	2-6" Cs × 15.5-lb. 2 Pl. 8"×1/2" 1/2" Rivets	9.12 8.00 17.12	2.33	120 85 50	23'3 5/8" 16'6 1/16" 9'8 1/2"
2-5" Cs × 6.5-lb. 2 Pl. 9 1/2"×1/4"  1/2" Rivets	3.90 4.75 8.65	2.31	120 85 50	23'1 13/16" 16'6 3/4" 9'9"	2 Cs 5"×11.5-lb. 2 Pl. 9 1/2"×1/2" 1/2" Rivets	6.76 9.50 16.26	2.39	120 85 50	23'10 13/16" 16'11 1/4" 9'11 1/2"
2 Cs 8"×11 1/4-lb. 1 I 8"×18.0-lb  5/8" Rivets	6.70 5.33 12.03	2.38	120 85 50	23'9 5/8" 16'10 5/16" 9'11"	2 Cs 8"×18.75-lb. 1 I 8"×20.50-lb. 5/8" Rivets	11.02 6.03 17.05	2.32	120 85 50	23'2 7/16" 16'5 5/16" 9'8"

Light Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.
1 I 10″ x 25-lb. 2 Pl. 11″ x 5/16″  5/8″ Rivets	7.37 6.88 14.25	2.31	120 85 50	23′1 3/16″ 16′4 3/8″ 9′7 1/2″	1 I 10″ x 35-lb. 2 Pl. 11″ x 5/8″ 5/8″ Rivets	10.29 13.75 24.04	2.47	120 85 50	24′8 3/8″ 17′5 15/16″ 10′3 1/2″
4-5″ Bulb L^s x 10.1-lb. 1 Pl. 6″ x 5/16″  5/8″ Rivets	11.88 1.88 13.76	2.48	120 85 50	24′9 5/8″ 17′6 13/16″ 10′4″	4-5″ Bulb L^s x 13.5-lb. 1 Pl. 6″ x 7/16″ 5/8″ Rivets	15.88 2.63 18.51	2.42	120 85 50	24′2 3/8″ 17′1 11/16″ 10′1″
4-4″ Z's x 1/4″ 1 Pl. 7″ x 1/4″  3/4″ Rivets	9.64 1.75 11.39	2.46	120 85 50	24′7 3/16″ 17′5 1/8″ 10′3″	4-4″ Z's x 5/8″ 1 Pl. 7″ x 5/8″ 3/4″ Rivets	22.20 4.38 26.58	2.51	120 85 50	25′1 3/16″ 17′9 3/8″ 10′5 1/2″
2 Pl. 9″ x 1/4″ 4 L s 2″ x 2″ x 1/4″ 2 Pl. 5 1/4″ x 1/4″  1/2″ Rivets	4.500 3.760 2.625 10.885	2.33	120 85 50	23′3 5/8″ 16′6 1/16″ 9′8 1/2″	2 Pl. 9″ x 7/16″ 4 L s 2″ x 2″ x 7/16″ 2 Pl. 5 1/4″ x 7/16″ 1/2″ Rivets	7.88 6.24 4.59 18.71	2.34	120 85 50	23′4 13/16″ 16′6 3/8″ 9′9″

General Notes:
Rivet Spacing = 4 diameters of rivet
End distance = 2 " " "
Make three columns of each length for all shapes

PROGRESS REPORT OF SPECIAL COMMITTEE ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.

To the American Society of Civil Engineers.

Gentlemen:—Your Special Committee on "Bituminous Materials for Road Construction" respectfully submits the following Progress Report:

Your Committee has especially considered during the past year the various methods of writing specifications covering the physical and chemical properties of bituminous materials for use in the construction of bituminous surfaces and bituminous pavements.

Many of these properties vary to a remarkable degree, dependent primarily upon the source of the material and the methods employed in refining. It is recognized that it is often essential to specify narrow limitations of certain properties in order to secure desired chemical and physical characteristics and uniformity in the manufactured material. It is not in many cases practicable to specify the same limitations, except for materials obtained from the same or similar sources and prepared in the same manner.

Therefore it is suggested that, for the present at least, whenever comprehensive specifications are to be prepared so as to admit a variety of types of materials, separate specifications as may be necessary be prepared for each type. As an illustration, the specification for the bituminous cement to be used in the construction of a bituminous pavement by the mixing method might contain independent specifications covering, within narrow limits, the physical and chemical properties of each of the following bituminous materials: refined water-gas tars, refined coal-gas tars, mixtures of tars, asphalts containing native bitumen from one or more sources, asphalts obtained by refining petroleum from one or more sources, and asphalts which are solid or semi-solid compounds composed of the bitumens mentioned with petroleums or derivatives thereof.

Your Committee, recognizing an unfortunate tendency to use as the generic expression the terms "bituminous material" and "bitumen" synonymously, recommends that the term "bituminous material" be used as a generic expression when referring to road and paving materials containing bitumen, and that the term "bitumen" be used in a restricted sense as covered by the following definition adopted by the American Society for Testing Materials:

"Bitumens are mixtures of native or pyrogenous hydrocarbons and their non-metallic derivatives, which may be gases, liquids, viscous liquids, or solids, and which are soluble in carbon disulphide."

An illustration of ambiguity in the use of the two terms may be found in some descriptions of bituminous pavements, where a bitu-

minous cement, only 75% soluble in carbon disulphide, was used. In the general description of these pavements, the word "bitumen" was used to refer to the material as a whole, while, in giving the analysis of the mix, the word "bitumen" was used to refer only to that portion of the cement soluble in carbon disulphide.

Your Committee, in its Report at the last Annual Meeting of the Society, recommended then that especial attention be directed during the year 1912 to the following points:

(*a*)—The maximum amount of distillate from tars coming over, up to 170° cent., that can satisfactorily be allowed;

(*b*)—The maximum percentage of free carbon that can be successfully allowed in a tar used for superficial treatment under varying climatic conditions;

(*c*)—The maximum amount of residue and the limitations of penetration thereof that can be satisfactorily permitted in an asphaltic oil used for superficial treatment.

Your Committee then recommended, also, that general consideration be given to

(*d*)—The use of a seal coat on bituminous pavements constructed by the mixing method; and

(*e*)—The establishment of standards for arriving at figures of cost on engineering work, especially road work.

Taking these points *seriatim,* your Committee, having given them especial attention during the past year, now agrees as follows:

(*a*)—The maximum amount of distillate, up to 170° cent., allowable in tars may depend upon the proportions of the other fractions in a tar, or upon the conditions of use of the tar, or upon the conditions on the road surface after its construction, or possibly upon some other factors. Although the Committee has endeavored to secure all possible information, it feels that certain important data are still lacking, and therefore deems it best to refrain now from stating what might seem to be too hasty conclusions in the matter.

(*b*)—Your Committee submits that considerable evidence seems to make undesirable in tars for superficial treatments the presence of free carbon beyond a maximum, dependent on the fraction distillable between 170 and 300° cent., in order at least to avoid brittleness at low temperatures.

(*c*)—Your Committee agrees that the maximum amount of residue allowable in an asphaltic oil used for superficial treatment should be as great as permitted by the physical conditions of its use, subject to certain limits in the penetration of this residue, and that such limits in penetration should be:

A minimum which will ensure against excessive hardness or slipperiness under travel, at low temperatures.

A maximum which will ensure against excessive softness or lack of body at the highest temperatures to be expected after use on the road.

(*d*)—Your Committee has, as it recommended last year, devoted considerable attention to the use of a seal coat on mixed bituminous pavements, but simply wishes at this time to recommend further investigation concerning the use of seal coats on all types of bituminous pavements.

(*e*)—Your Committee renews its recommendation of last year regarding the establishment of standards for the record and compilation of cost figures.

Your Committee also last year in its Report advocated the adoption of two expressions, *viz.,* "Bituminous Surfaces" and "Bituminous Pavements", to cover the ordinary uses of bituminous materials in road construction, and defined the application of each. It now wishes to report that these terms, as defined by the Committee, seem to have been widely adopted, with general benefit. Your Committee now further advocates the adoption of the following definition:

"Bituminous Concrete Pavements" are those composed of stone, gravel, sand, shell, or slag, or combinations thereof, and bituminous materials incorporated together by mixing methods.

Very respectfully,

W. W. Crosby, *Chairman,*
Hubert K. Bishop,
Arthur W. Dean,
Arthur H. Blanchard, *Secretary.*

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

March 5th, 1913.—8.30 P. M.—A regular business meeting will be held, and a paper by Caleb Mills Saville, M. Am. Soc. C. E., entitled "Hydrology of the Panama Canal," will be presented for discussion.

This paper was printed in *Proceedings* for January, 1913.

March 19th, 1913.—8.30 P. M.—At this meeting a paper by E. J. Schneider, M. Am. Soc. C. E., entitled "Construction Problems, Dumbarton Bridge, Central California Railway," will be presented for discussion.

This paper was printed in *Proceedings* for January, 1913.

April 2d, 1913.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "Shearing Strength of Construction Joints in Stems of Reinforced Concrete T-Beams, as Shown by Tests," by Lewis J. Johnson, M. Am. Soc. C. E., and John R. Nichols, Jun. Am. Soc. C. E.; and "Fremantle Graving Dock: Steel Dam Construction for North Wall," by Joshua Fielden Ramsbotham, Assoc. M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Gavin N. Houston, M. Am. Soc. C. E., 409 Equitable Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, and, until further notice, will take place at the Colorado Traffic Club.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meetings)

December 14th, 1912.—The meeting was called to order; President Ketchum in the chair; G. N. Houston, Secretary; and present, also, 18 members and 5 guests.

A paper entitled "Proposed Legislation" was presented by C. W. Comstock, M. Am. Soc. C. E., after which Mr. E. F. Bohm, of the *Great West Magazine,* reviewed the irrigation situation in the West. A general discussion of the subject followed.

The proposed bill for licensing engineers in Colorado was discussed, and, on motion, was referred to the Committee which had it in charge during the past year, with instructions to introduce it again, without change, in the next Legislature. On request President Ketchum's place on the Committee was taken by Secretary Houston.

A general discussion followed, in which nearly all present took part.

Adjourned.

January 11th, 1913.—The meeting was called to order; President Ketchum in the chair; G. N. Houston, Secretary; and present, also, 16 members and 7 guests.

Messrs. G. G. Anderson, J. C. Ulrich, and H. S. Crocker were appointed an Auditing Committee.

Messrs. Anderson, Comstock, Field, Crocker, and Houston, were appointed a Committee on Legislation to look into proposed legislation affecting engineering practice in general, and make such suggestions as they deem proper.

A paper entitled "The Prewitt Irrigation Proposition" was presented by J. C. Ulrich, M. Am. Soc. C. E., who illustrated his remarks with lantern slides.

Adjourned.

Atlanta Association

On March 14th, 1912, the Atlanta Association of Members of the American Society of Civil Engineers was organized, with the following officers: Arthur Pew, President; William A. Hansell, Jr., Secretary; and Messrs. James N. Hazlehurst and Alexander Bonnyman, Members of the Executive Committee. The Association will hold its meetings in the house of the University Club.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 302 Board of Trade Building, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From January 2d to February 1st, 1913)

DONATIONS*

THE ECONOMICS OF RAILROAD CONSTRUCTION.

By Walter Loring Webb, M. Am. Soc. C. E. Second Edition, Revised. Cloth, 8¼ x 5¼ in., illus., 10 + 347 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1912. $2.50.

The first edition of this work was published in 1906. Since that time the Interstate Commerce Commission has made changes in the classification of operating expenses, which, it is stated, necessitated a practical rewriting of the chapters on that subject, as well as those on Distances, Curvature, and Grade. The changes in the first chapters consist principally of enlarging tables and diagrams and of bringing them up to date. As in the previous edition, this book is written, it is stated, from the standpoint of the constructing or operating engineer, railroad legislation and kindred subjects being considered only as they affect questions which must be answered by the railroad engineer, namely, legal methods of organization, railroad finance, etc. The Contents are: Part I, Financial and Legal Elements of the Problem: Railway Statistics; The Organization of Railroads; Capitalization; The Valuation of Railroad Property; Estimation of Volume of Traffic. Part II, Operating Elements of the Problem: Operating Expenses; Motive Power; Economics of Car Construction; Track Economics; Train Resistance; Momentum Grades. Part III, Physical Elements of the Question: Distance; Curvature; Minor Grades; Ruling Grades; Pusher Grades; Balancing Grades for Unequal Traffic; Index.

A HANDBOOK OF ENGLISH FOR ENGINEERS.

By Wilbur Owen Sypherd. Leather, 7 x 4¾ in., 314 pp. Chicago and New York, Scott, Foresman and Company, 1913. $1.50.

This book is designed primarily, it is stated, to provide a manual of technical English and composition for advanced engineering students and for young engineers in actual practice, together with specific instruction in the writing of business letters, reports, instructions, estimates, specifications, and contracts, and articles for technical journals. Chapter I is devoted to the specific discussion of the planning and presentation of technical papers, kinds of writing, language, form, etc., and, for the purposes of analysis and illustration, may be used in connection with Chapter V, which contains specimens of technical articles from various fields of engineering. In Chapter II are given rules for the use of abbreviations, hyphens, numbers, capital letters, etc., which are said to be in accord with the best practice in engineering writing. Business letters and reports are discussed in Chapter III, particular stress being laid, it is stated, on the writing of letters of application. In Chapter IV, the author discusses the various kinds of reports, and examples of reports on laboratory experiments, tests, progress work, inspection work, etc., are given. The Appendices contain directions for proof-reading, examples of faulty paragraphs and sentences, lists of words frequently misused and misspelled, and a bibliography of the subjects treated in the book. The Contents are: The General Problems of Engineering Writing; Mechanical Details Common to the Various Forms of Technical Writing; Business Letters; Reports; Articles for Technical Journals; Index.

AMERICAN CIVIL ENGINEERS' POCKET BOOK.

By Mansfield Merriman, M. Am. Soc. C. E., Editor-in-Chief; Messrs. Ira O. Baker, Arthur H. Blanchard, Charles B. Breed, Walter J. Douglas, Louis A. Fischer, George A. Goodenough, Allen Hazen, Frank P. McKibben, Edward R. Maurer, Rudolph P. Miller, Alfred Noble, Frederick E. Turneaure, Walter Loring Webb, Gardner S. Williams, Associate Editors. Second Edition, Enlarged. Leather, 7 x 4¼ in., illus., 8 + 1473 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1912. $5.00.

In the preface to the first edition the fundamental principles observed in the preparation of this book are stated to have been: (1) The selection of topics to which engineers desire to refer; (2) The condensation of the matter so that the

*Unless otherwise specified, books in this list have been donated by the publishers.

greatest amount may be put in the assigned space and, at the same time, be clearly represented; (3) The fact that this pocket-book must be better and fuller than any hitherto published in the English language. The former arrangement of the subject-matter is followed in this edition. Two new sections have been added, as well as much new matter in the case of the other sections, and the editor hopes that, in its enlarged form, the book may advance sound engineering practice and assist the younger members of the Profession to a greater extent than before. The Contents are: Mathematical Tables, by Mansfield Merriman; Surveying, Geodesy, Railroad Location, by Charles B. Breed; Steam and Electric Railroads, by Walter Loring Webb; Materials of Construction, by Rudolph P. Miller; Plain and Reinforced Concrete, by Frederick E. Turneaure; Masonry, Foundations, Earthwork, by Ira O. Baker; Masonry and Timber Structures, by Walter J. Douglas; Steel Structures, by Frank P. McKibben; Hydraulics, Pumping, Water Power, by Gardner S. Williams; Water Supply, Sewerage, Irrigation, by Allen Hazen; Dams, Aqueducts, Canals, Shafts, Tunnels, by Alfred Noble and Silas H. Woodard; Mathematics and Mechanics, by Edward R. Maurer; Physics, Meteorology, Weights and Measures, by Louis A. Fischer; Steam and Electric Engineering, by George A. Goodenough and F. Malcolm Farmer; Highways and Streets, by Arthur H. Blanchard; Index, by Clinton L. Bogert.

EARTHWORK AND ITS COST.

By Halbert Powers Gillette, M. Am. Soc. C. E. Second Edition. With a Chapter on Ditching and Trenching Machinery, by E. E. R. Tratman, Assoc. M. Am. Soc. C. E. Cloth, 9½ x 6¼ in., illus., 11 + 238 pp. New York and London, McGraw-Hill Book Company, 1912. $2.00.

It is stated that it is only through a knowledge of cost details, such as are obtained from a study of tools and methods of construction, that an engineer can hope to design economic structures and predict the actual cost with accuracy, and the author's principal aim in this book has been, it is said, to formulate rules and lay down principles to be used by engineers and contractors in estimating such costs. The Chapter headings are: Introduction; Earth Shrinkage; Earth Classification and Factors in Estimating Earthwork; Cost of Loosening and Shoveling; Cost of Dumping, Spreading, Rolling, etc., Cost of Wheelbarrows and Carts; Cost by Wagons; Cost by Buck and Drag Scrapers; Cost by Wheel Scrapers; Cost by Elevating Grader; Cost by Steam Shovel; Cost by Cars; How to Handle a Steam Shovel Plant; Summary and Table of Costs; The Cost of Hydraulic Excavation; Cost of Dredging; Miscellaneous Cost Data; Earth and Earth Structures; Appendix A, Rapid Field and Office Survey Work; Appendix B, Overhaul Calculation; Appendix C, A Small "Home-Made" Dipper Dredge or Steam Shovel; Appendix D, Ditching and Trenching Machinery, by E. E. R. Tratman; Index.

RAILROAD CONSTRUCTION.

By Charles Lee Crandall, M. Am. Soc. C. E., and Fred Asa Barnes, Assoc. M. Am. Soc. C. E. Cloth, 9½ x 6¼ in., illus., 8 + 321 pp. New York and London, McGraw-Hill Book Company, 1913. $3.00.

The preface states that this book had its beginning in some notes on railroad construction which were issued in mimeograph form for the use of students in the College of Civil Engineering at Cornell University about twenty-five years ago. These notes have been frequently revised and brought up to date and are now presented in book form for general use. The subject-matter deals directly with construction work and includes descriptions of methods, materials, plant, costs, etc. At the end of each chapter references to books dealing with the subject discussed are given, which, it is hoped, will stimulate the student to do outside reading and aid the young engineer in obtaining special information as to handling work. The Chapter headings are: Introductory; Earthwork; Rock Excavation; Tunneling; Masonry; Foundations; Culvert and Bridge Masonry; Trestles and Bridges; Track Material and Roadbed; Estimates and Records; Index.

Gifts have also been received from the following:

Am. Mathematical Soc. 1 pam.
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Am. Water-Works Assoc. 1 bound vol.
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Institution of Gas Engrs. 1 bound vol.
Institution of Min. and County Engrs. 1 bound vol.
Institution of Min. and Metallurgy. 1 bound vol.
Institution of Min. Engrs. 1 pam.
Institution of Naval Archts. 1 bound vol.
Jefferds, Moses R. 1 vol., 5 maps.
Kansas-State School of Mines and Metallurgy. 1 pam.
Kelly, C. W. 1 bound vol., 9 pam.
Koninklijk Instituut van Ingenieurs. 1 pam.
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New Jersey-State Board of Assessors. 2 bound vol., 1 pam.
New London, Conn.-Water and Sewer Dept. 1 pam.
New York City-Commr. of Public Works. 1 pam.
New York State-Public Service Comm., Second Dist. 6 pam.
New York-State Dept. of Health. 1 bound vol.
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New York State Coll. of Forestry. 1 pam.
Ohio-Auditor of State. 2 vol.
Ohio-Chief Insp. of Mines. 1 bound vol.
Ohio-Gen. Assembly. 1 bound vol.
Ohio-Geol. Survey. 1 bound vol.
Ohio-State Library. 1 pam.
Ohio Soc. of Mech., Elec. and Steam Engrs. 8 pam.
Ohio State Univ. 1 vol., 1 pam.
Oklahoma-Corporation Comm. 1 bound vol.
Oklahoma-Geol. Survey. 1 pam.
Ontario, Canada-Bureau of Mines. 1 vol.
Oregon-State Board of Health. 1 pam.
Oregon-State Forester. 1 pam.
Panama R. R. Co. 1 pam.
Pennsylvania-Bureau of Industrial Statistics. 1 bound vol.
Pennsylvania-Dept. of Forestry. 1 bound vol.
Permanent Inter. Assoc. of Navigation Congresses. 1 bound vol., 22 pam.
Philadelphia, Pa.-Bureau of Surveys. 1 bound vol.
Philadelphia, Pa.-Mayor. 3 bound vol.
Philippine Islands-Bureau of Forestry. 1 pam.
Philippine Islands-Bureau of Sci. 1 pam.
Porto Rico-Commrs. of Agri. 1 pam.
Providence, R. I.-Dept. of Public Works. 1 pam.
Ry. Signal Assoc. 1 pam.
Reibling, W. C. 2 pam.
Rhoades, John Hansen. 1 pam.
Rhode Island-Board of Harbor Commrs. 1 pam.
Rhode Island-Board of Public Roads. 1 bound vol.
Rhode Island-Commr. of Forestry. 1 pam.
Richmond, Va.-Special Accountant. 6 bound vol.
Salem, Mass.-City Plans Comm. 1 pam.
Smithsonian Institution. 2 bound vol.

Soc. of Naval Archts. and Marine Engrs. 1 pam.
Soc. of Ry. Club Secretaries. 1 pam.
South Dakota-Bureau of R. R. Commrs. 1 pam.
Tennessee-Min. Dept. 1 bound vol.
Tennessee-State Geol. Survey. 10 pam.
Texas-R. R. Comm. 1 vol.
Texas & Pacific Ry. Co. 1 pam.
Tokyo Imperial Univ. 1 vol.
U. S.-Bureau of Mines. 5 pam.
U. S.-Bureau of Standards. 2 pam.
U. S.-Bureau of the Census. 1 pam.
U. S.-Chf. of Engrs. 2 bound vol., 19 specif.
U. S.-Commr. of Lighthouses. 1 pam.
U. S.-Library of Congress. 1 bound vol., 1 pam.
U. S.-Office of Public Roads. 2 pam.
U. S.-Supt. of Yellowstone National Park. 1 pam.
Utah-Agri. Exper. Station. 6 pam.
Virginia, Univ. of. 1 vol.
Washington-Commr. of Public Lands. 1 vol.
Washington-Public Service Comm. 1 vol., 1 pam.
Washington-State Library. 1 pam.
Western Australia-Dept. of Mines. 1 vol.
White, W. H. 1 pam.
Wisconsin-Insp. of Illuminating Oils. 1 pam.
Wisconsin-R. R. Comm. 2 pam.
Wyoming-State Engr. 1 pam.
Wyoming-State Geologist. 1 pam.
Yonkers, N. Y.-Bureau of Water. 1 pam.

BY PURCHASE

A Handbook of Sugar Analysis: A Practical and Descriptive Treatise for Use in Research, Technical and Control Laboratories. By C. A. Browne. John Wiley & Sons, New York; Chapman & Hall, Limited, London, 1912.

The Bacteriology of Surface Waters in the Tropics. By Wm. Wesley Clemesha. E. & F. N. Spon, Ltd., London; Thacker, Spink & Co., Calcutta, 1912.

Preliminary Studies in Bridge Designs, Being the First of a Series of Small Volumes, Each Complete in itself, Dealing with the Design of Ordinary Highway Bridges of Moderate Spans. By Reginald Ryves. The St. Bride's Press, Ltd., London.

Introduction to the Chemistry and Physics of Building Materials. By Alan E. Munby. Archibald Constable & Co., Ltd., London, 1909.

Analysis of Metallurgical and Engineering Materials: A Systematic Arrangement of Laboratory Methods. By Henry Wysor. The Chemical Publishing Co., Easton, Pa.; Williams & Norgate, London, 1912.

Railroads, Rates and Regulation. By William Z. Ripley. Longmans, Green and Co., New York and London, 1912.

A Dictionary of Applied Chemistry. By *Sir* Edward Thorpe, Assisted by Eminent Contributors. Vol. 3. Revised and Enlarged Edition. Longmans, Green and Co., New York and London, 1912.

The Prevention of Malaria. By Roland Ross and others. E. P. Dutton & Company, New York, 1910.

Steam Economy in the Sugar Factory. By Karl Abraham. Translated from the German Edition by E. J. Bayle. John Wiley & Sons, New York; Chapman & Hall, Limited, London, 1912.

Le Genie Civil: 2e Table Generale des Matieres; Tomes 21 a 40 (1892-1902). C. H. Talansier, Administrateur-Délégué du Génie Civil, Paris.

Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens insbesondere aus den Laboratorien der technischen Hochschulen. Herausgegeben vom Verein deutscher Ingenieure. Hefte 125-128. Julius Springer, Berlin, 1912.

Royal Commission on Sewage Disposal: Eighth Report of the Commissioners Appointed to Inquire and Report What Methods of Treating and Disposing of Sewage (Including any Liquid from any Factory or Manufacturing Process) May Properly be Adopted. Vol. I. Report. Published by His Majesty's Stationery Office, London. 1912.

The World Almanac and Encyclopedia, 1913. The Press Publishing Co., New York.

A Manual of Machine Drawing and Design. By David Allan Low and Alfred William Bevis. New Edition, Revised and Enlarged. Longmans, Green and Co., New York and London, 1910.

Lieber's Standard Telegraphic Code. By B. Franklin Lieber. The Lieber Code Co., New York, 1896.

The A B C Universal Commercial Electric Telegraphic Code, Specially Adapted for the Use of Financiers, Merchants, Shipowners, Underwriters, Engineers, Brokers, Agents, etc. By W. Clausen-Thue. Fifth Edition. American Code Company, New York, 1901.

Brooklyn Daily Eagle Almanac, 1913: A Book of Information, General of the World, and Special of New York City and Long Island. Brooklyn Daily Eagle, Brooklyn, N. Y.

Notes on Hydraulics: A Pocket Book of Useful Data for Engineers, Architects, Factory Managers, Fire Insurance Inspectors, and Students. By Ira J. Owen. The Insurance Press, New York.

Water: Its Purification and Use in the Industries. By William Wallace Christie, M. Am. Soc. C. E. D. Van Nostrand Company, New York, 1912.

Gas Works Directory and Statistics, 1912-13, With a List of Chairmen, Managers, Engineers, and Secretaries and Lists of Associations of Engineers and Managers. Hazell, Watson and Viney, London, 1912.

Royal Commission on Canals and Waterways, Vol. 1-9, 11-12. Printed for His Majesty's Stationery Office, Wyman & Sons, Ltd., London, 1906-11.

SUMMARY OF ACCESSIONS

(From January 2d to February 1st, 1913)

Donations (including 41 duplicates)	317
By purchase	34
Total	351

MEMBERSHIP

ADDITIONS

(From January 3d to February 6th, 1913)

MEMBERS		Date of Membership.
BUDD, RALPH. Aberdeen Hotel, St. Paul, Minn		Jan. 7, 1913
CARUTHERS, WILLIAM STODDERT. Div. Engr., California Highway Comm., 1416 O St., Sacramento, Cal		Jan. 7, 1913
CONWAY, JOHN SEBASTIAN. Deputy Commr. of Lighthouses, Washington, D. C	Assoc. M.	May 4, 1909
	M.	Jan. 7, 1913
DONLEY, WILLIAM MCCLURG. Borough Engr., Boroughs of Carrick, Mt. Oliver, St. Clair, and Knoxville, Mt. Oliver P. O. Station, Pittsburgh, Pa		Jan. 7, 1913
DOTY, JOHN WILLIAMS. With The Foundation Co., 115 Broadway, New York City	Jun.	Nov. 3, 1903
	Assoc. M.	Jan. 4, 1905
	M.	Jan. 7, 1913
FISHER, HOWELL TRACY. Tunnel Engr., Mackenzie, Mann & Co., Ltd., 411 Dorchester St., West, Montreal, Que., Canada	Assoc. M.	Nov. 1, 1905
	M.	Jan. 7, 1913
HAMMER, JOHANNES MARCELIUS. Asst. Engr., Isthmian Canal Comm., 5712 Forbes St., Pittsburgh, Pa		Jan. 7, 1913
HANNA, FRANK WILLARD. Engr., U. S. Reclamation Service, Washington, D. C		Jan. 7, 1913
HATT, WILLIAM KENDRICK. Prof. of Civ. Eng., and Director, Testing Labaratory, Purdue Univ., Lafayette, Ind	Assoc. M.	June 4, 1902
	M.	Jan. 7, 1913
HUNICKE, WILLIAM AUGUST. Chf. Engr., Louisiana & Northwest Ry., Homer, La.	Assoc. M.	Mar. 2, 1904
	M.	Jan. 7, 1913
LAIRD, HARRY SNEDDEN. Res. Engr., Am. Water-Works & Guarantee Co., Hydro-Elec. Dept., Cheat Haven, Pa		Dec. 3, 1912
MACCORNACK, CLYDE WEBSTER. Asst. Chf. Engr., Phœnix Bridge Co., 324 Gay St., Phœnixville, Pa	Jun.	Jan. 31, 1905
	Assoc. M.	Dec. 1, 1908
	M.	Jan. 7, 1913
NEWMAN, JEROME. Chf. Engr., Board of State Harbor Commrs., 18 Ferry Bldg., San Francisco, Cal		Jan. 7, 1913
ROBINSON, HOLTON DUNCAN. Cons. Engr., 357 West 121st St., New York City	Jun.	Mar. 1, 1893
	Assoc. M.	Jan. 3, 1894
	M.	Jan. 7, 1913
YATES, WILLIAM HENRY. Superv. Engr., N. Y. State Barge Canal, Barge Canal Office, Albany, N. Y	Jun.	Feb. 2, 1904
	Assoc. M.	Jan. 8, 1908
	M.	Jan. 7, 1913

ASSOCIATE MEMBERS		
BLACK, ROGER DERBY. Capt., Corps of Engrs., U. S. A., U. S. Engr. Office, 25 North Pearl St., Albany, N. Y	Jun.	Jan. 3, 1905
	Assoc. M.	Jan. 7, 1913

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
BOCK, CARL AUGUST. Locating Engr., Caribbean Constr. Co., St. Marc, Haiti......	Jun. Assoc. M.	Aug. 31, 1909 Dec. 3, 1912
BROWN, AMON BENJAMIN. Asst. Engr., U. S. Reclamation Service, Rupert, Idaho...........................		Jan. 7, 1913
BROWN, LEVANT R. Asst. Engr., Bureau of Public Works, Manila, Philippine Islands......................		Jan. 7, 1913
BURROUGHS, FREDRIC SIDNEY. Asst. Engr., Public Service Comm. of Washington, Olympia, Wash............		Dec. 3. 1912
BURTON, WAYNE JOSEPH. Designing Engr., Office of Chf. Engr., M. of W., Mo. Pac. Ry., 627 Midland Bldg., Kansas City, Mo..........................	Jun. Assoc. M.	Sept. 4, 1906 Jan. 7, 1913
CAMERON, KENNETH MACKENZIE. Prin. Asst. to Asst. Chf. Engr., Dept of Public Works, 132 Flora St., Ottawa, Ont., Canada............................	Jun. Assoc. M.	Oct. 6, 1903 Oct. 1, 1912
COMBER, STAFFORD XAVIER. Underpinning & Foundation Co., 290 Broadway, New York City...............		Jan. 7, 1913
CROCKER, FOSTER BALDWIN. Barge Canal Office, R. F. D. No. 5, Rome, N. Y......	Jun. Assoc. M.	June 30, 1911 Jan. 7, 1913
DEUTSCHBEIN, HARRY JOHNSON. Asst. Supt., The Foundation Co., Little Hocking, Ohio..............................	Jun. Assoc. M.	Dec. 6, 1904 Jan. 7, 1913
DIBBLE, BARRY. Electrical Engr., U. S. Reclamation Service, Minidoka, Idaho...........................		Dec. 3, 1912
DORON, CHARLES SLAUTER. Engr. in Chg., Dept. of Parks, Brooklyn, N. Y...............................		Jan. 7, 1913
DRIGGS, EDWIN LEROY. Asst. Engr., Bureau of Public Works, Manila, Philippine Islands..............................	Jun. Assoc. M.	Nov. 5, 1907 Oct. 29. 1912
FITZSIMONS, WAVELAND SINCLAIR. Junior Engr., U. S. Engr. Office, Georgetown, S. C....................		Oct. 29, 1912
GILLELEN, FRANK. (Olmsted & Gillelen), 604 Wright & Callender Bldg., Los Angeles. Cal.................................	Jun. Assoc. M.	May 1, 1906 Feb. 4, 1913
GIQUEL, RAFAEL SANCHEZ. First Engr. in Chg. of Design of Aqueducts for Province of Havana, Calle A, No. 172, Vedado, Havana, Cuba................	Jun. Assoc. M.	Feb. 4, 1908 Dec. 3, 1912
GREGG, JOHN HUSTON CLARK. Asst. Engr., Carpenter & Boxley & Herrick, Inc., 6720 Ridge Boulevard, Brooklyn, N. Y.................................		Jan. 7, 1913
HALL, JULIUS REED. Designing Engr., Strauss Bascule Bridge Co., 608 Monroe Bldg., Chicago, Ill.........................	Jun. Assoc. M.	June 6, 1905 Dec. 3, 1912

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
HASELTON, GAGE. 247 Stout St., Portland, Ore.	Jun. / Assoc. M.	May 3, 1904 / Jan. 7, 1913
HUSBAND, CHARLES MARSH. Civ. Engr., Pressed Steel Car Co., Pittsburgh. (Res., 412 Division St., Bellevue), Pa.		Jan. 7, 1913
KIRKPATRICK, RALPH ZENAS. Junior Engr., care, Chf. Engr.'s Office, Isthmian Canal Comm., Pedro Miguel, Canal Zone, Panama		Dec. 3, 1912
KORSMO, AMUND MARIUS. Asst. Engr., The Faris Eng. & Constr. Co., Oakley, Idaho		Jan. 7, 1913
LEACH, THOMAS. Designer, Dominion Bridge Co., Ltd., Montreal, Que., Canada	Jun. / Assoc. M.	June 30, 1910 / Dec. 3, 1912
LINDSAY, RICHARD LEE. 502 Fourteenth Ave., Roanoke, Va.	Jun. / Assoc. M.	Sept. 4, 1906 / Sept. 3, 1912
MACLEAN, WILLIAM EUSTACE. Care. City Clerk, Salmon Arm, B. C., Canada		Jan. 7, 1913
MCKINNEY, FRANCIS WILLIAM. Asst. Engr., Baltimore Sewerage Comm., American Bldg., Baltimore, Md.	Jun. / Assoc. M.	Mar. 31, 1908 / Jan. 7, 1913
MOSS, WILLIAM BENJAMIN. Junior Engr., U. S. Engr. Dept. at Large, Room 807, Army Bldg., New York City (Res., 36 Pommer Ave., Tompkinsville, N. Y)		Jan. 7, 1913
MUMM, HANS, JR. County Engr., Snohomish County (Res., 2216 Hoyt Ave.), Everett, Wash.		July 9, 1912
MURPHY, JAMES FRANCIS. 165 Remsen St., Brooklyn, N. Y.	Jun. / Assoc. M.	May 31, 1910 / Sept. 3. 1912
PHELPS, HOWARD EASTWOOD. City Engr. and Bldg. Insp., Boulder, Colo.		Jan. 7, 1913
RENNER, CHARLES JOSEPH. City Engr. and Supt., Water, Streets, and Sewers, St. Albans, Vt.		Dec. 3, 1912
SMITH, PLUMER HENRY. Chf. Engr., Texas City Terminal Co., Texas City, Tex.		Jan. 7, 1913
STANAGE, JOHN LYNCH. Prin. Asst. Engr., Board of Engrs., Fort Worth Water Supply. Fort Worth, Tex.		Jan. 7, 1913
VAN SCOYOC, HARRY STEWART. Insp. Engr., Canada Cement Co., Ltd.. Herald Bldg., Montreal, Que., Canada.		Jan. 7, 1913
WATTERS, VERNON GREGG. Chf. Engr.. Live Oak. Perry & Gulf R. R., Live Oak, Fla.		Jan. 7, 1913
WILLIS, ALBERT JONES. Instr. in Civ. Eng., Cooper Union (Res., 1019 Trinity Ave.), New York City	Jun. / Assoc. M.	June 4, 1907 / Dec. 3, 1912

ASSOCIATES

		Date of Membership.
DAVIS, FREDERICK CALVIN. Mgr., Pacific Clay Products Publicity Bureau, 515 Buena Vista Ave., San Francisco, Cal.		Jan. 7, 1913

ASSOCIATES (*Continued*)	Date of Membership.
HILL, ROBERT. 2 Toronto St., Toronto, Ont., Canada......	July 9, 1912
PARSHALL, RALPH LEROY. 926 Akin Ave., Fort Collins, Colo......	Jan. 7, 1913

JUNIORS

ARNOLT, FRITZ MUSS. New York Univ., University Heights, New York City......	Dec. 3, 1912
BAILEY, PAUL. Res. Engr., Haviland & Tibbetts, 515 Ninth St., Sacramento, Cal......	Jan. 7, 1913
BAKER, ALBERT ASA. Civ. Engr., U. S. N., Navy Yard, Brooklyn, N. Y......	Oct. 29, 1912
BURR, GEORGE LINDSLEY. 416 High St., Keokuk, Iowa......	Jan. 7, 1913
DE JONGH, ARTHUR FRANCIS. Banes, Oriente, Cuba......	Dec. 3, 1912
EASON, FRANK GARY. U. S. Drainage Engr., Drainage Investigations, U. S. Dept. of Agri., Washington, D. C......	July 9, 1912
JONES, PAUL SIDNEY. Asst. Engr., Co-operative Irrig. Investigations, Colo. Agri. Experiment Station and U. S. Dept. of Agri., Fort Collins, Colo......	Dec. 3, 1912
LECKLITER, WALTER HARLAN. Corning, Iowa......	Oct. 1, 1912
MCMILLAN, WILLIAM BRUCE. 411 South 11th St., San José, Cal......	Jan. 7, 1913
PARKER, HENRY BRACKETTE. Leveler, N. Y. State Highway Dept., 259 Hamilton St., Albany, N. Y......	Jan. 7, 1913
PORZELIUS, ALBERT FREDRICK. Asst. Engr., Arkansas Water Co., Little Rock, Ark......	Jan. 7, 1913
SEAMAN, DANIEL HENRY. Estimator and Designer, Hay Foundry & Iron Works, 774 South 19th St., Newark, N. J......	Dec. 3, 1912
SMITH, WALTER MICKLE, JR. Estimator and Computer, Noble & Woodard, 7 East 42d St., New York City......	Jan. 7, 1913
STARKWEATHER, ALFRED KENNETH. Instrumentman with Passaic Val. Sewerage Comm., 30 Oakland Ave., Bloomfield, N. J......	Jan. 7, 1913
WRIGHT, FREDERICK JOHN. First Asst. Engr. to County Engr. of Passaic County, 410 Van Houten St., Paterson, N. J......	Jan. 7, 1913
YOUMANS, GEORGE LELAND. Res. Engr., G. N. Ry., Winesap, Wash......	Jan. 7, 1913

REINSTATEMENTS

MEMBERS	Date of Reinstatement.
QUINBY, EDWIN RUFUS......	Jan. 15, 1913

RESIGNATIONS

ASSOCIATE MEMBERS

	Date of Resignation.
HARRINGTON, FRANCIS BURCHARD	Jan. 7, 1913
SCOTT, JOHN KUHN	Jan. 7, 1913

JUNIORS

REESE, GEORGE WASHINGTON	Jan. 7, 1913
WARNER, GLENN	Jan. 7, 1913

DEATHS

ATTERBURY, CHARLES DE LA PLANE. Elected Associate Member Nov. 8th, 1909; died June 3d, 1912.

GRAY, GEORGE EDWARD. Elected Member July 2d, 1873; Honorary Member June 5th, 1894; died January 1st, 1913.

HAMILTON, WILLIAM GASTON. Elected Member October 7th, 1868; died January 23d, 1913.

MACGREGOR, JOHN. Elected Associate June 2d, 1903; died January 21st, 1913.

WATSON, JAMES. Elected Fellow December 5th, 1872; died December, 1910.

WHEELER, EBENEZER SMITH. Elected Member November 7th, 1883; died January 5th, 1913.

Total Membership of the Society, February 6th, 1913, 6 797.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(January 2d to February 1st, 1913)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0,70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Elektrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1,60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.

(54) *Transactions*, Am. Soc. C. E., New York City, $4.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England, 6d.
(69) *Der Eisenbau*, Leipzig, Germany.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Mining and Engineering World*, Chicago, Ill., 10c.
(83) *Gas Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*. Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Mining Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga., 10c.

LIST OF ARTICLES

Bridges.

The Hapuawhenua Viaduct, New Zealand.* Frederick William Furkert. (63) Vol. 189.
The Making and Driving of Reinforced-Concrete Piles.* Gower Bonverie Raynor Pimm. (63) Vol. 189.
The South Main Pier of the Quebec Bridge.* (12) Dec. 27.
Concrete Bridges and Viaducts.* A. M. Wolf. (87) Jan.
The High-Level Bridge over the New Widened Lines: Willesden Junction, L. and U. W. R.* (21) Jan.
The Bridge of the Canadian Pacific Railway at Lachine, P. Q.* (96) Jan. 2.
The Hopatcong-Slateford Cut-Off.* C. W. Simpson. (15) Jan. 3.
Replacing the Cuyahoga River Draw Bridge.* (14) Jan. 4.
Construction of the Fifth Street Viaduct, Fitchburg, Mass.* (14) Jan. 4.

*Illustrated.

Bridges—(Continued).

Development of Patented Concrete Bridges.* Daniel B. Luten (Paper read before the Am. Soc. of Eng. Contractors.) (62) Jan. 6.
General Method Adopted for Constructing a 312 ft. Reinforced Concrete Arch Bridge at Larimer Ave., Pittsburgh, Pa. Charles H. McAlister. (86) Jan. 8.
A Large Cantilever Bridge in China.* (From the *Far Eastern Review.*) (13) Jan. 9.
Protective Coatings for Railway Bridge Floors. A. W. Carpenter. (Abstract of paper read before the Maintenance-of-Way Master Painters Assoc.) (13) Jan. 9.
New Bridges at Vancouver, British Columbia.* (12) Serial beginning Jan. 10.
The Beaver Bridge over the Ohio River.* Frank W. Skinner, M. Am. Soc. C. E. (11) Serial beginning Jan. 10.
Erection of Kentucky and Indiana Bridge.* (15) Jan. 10.
Municipal Bridge Approach, St. Louis.* S. W. Bowen. (13) Jan. 16.
Erection of Monongahela River Bridge.* (15) Jan. 17.
Proposed Long-Span Bridge over the Mersey. (14) Jan. 18.
New Bridge over the Mississippi River at St. Paul on the C. G. W. Ry.* (18) Jan. 18.
The Design of Various Types of Highway Bridges from the Standpoint of Modern Traffic.* F. H. Neff. (Paper read before the Am. Assoc. for the Advancement of Science.) (86) Jan. 22; (14) Feb. 1.
Maintenance of Great City Bridges.* (14) Jan. 25.
Broadway Bridge across the Oswego River and the New York Barge Canal at Fulton, N. Y.* (14) Jan. 25.
Modern Engineering in an Ancient Eastern Capital, New Pontoon Bridge over the Golden Horn, Constantinople.* F. C. Coleman. (19) Jan. 25.
The Red Deer River Bridge, Alberta Central Railway.* Hugh A. Lumsden. (96) Jan. 30.
A Famous English Chain Suspension Bridge.* (13) Jan. 30.
A Temporary Drawbridge of the Horizontal Rolling Type.* (13) Jan. 30.
A Rolling Lift Bridge 186 ft. Long.* (20) Jan. 30.
Lift Bridges over the Buffalo River.* Emile Low. (15) Jan. 31.
Method of Designing Arches, Kingshighway Viaduct, St. Louis.* (14) Feb. 1.
Le Pont Suspendu Rigide de Saint-Martin-d'Ardèche.* Moreau. (35) Jan.
Le Fonçage des Caissons de Fondation du Nouveau Pont de Quebec.* (33) Jan. 18.
Berechnung der Kippzapfen von Brückengelenken.* S. Bobrowsky. (69) Dec.
Mechanischer Antrieb der Drehbrücke über die Tote Weichsel bei Danzig.* Harprecht. (102) Jan. 1.
Gerüstbrücke zur Ueberführung der Gäubahn über die Vorort und Gütergleise Stuttgart-Ludwigsburg beim Hauptbahnhof Stuttgart.* Schaechterle. (78) Jan. 3.

Electrical.

Electrolysis from Stray Electric Currents.* Albert F. Ganz. (59) Vol. 32.
Synchronous and Asynchronous Reactance. J. Rezelman. (73) Serial beginning Dec. 27.
Directive Wireless Telegraphy.* F. Addey. (Abstract of paper read before the Institution of Post Office Elec. Engrs.) (73) Dec. 27.
Speed Control of Three-Phase Motors.* Sidney A. Simon. (Paper read before the West of Scotland Branch of the Assoc. of Min. Elec. Engrs.) (22) Dec. 27.
Study of the Relative Merits of the Various Types of Electric Arc and Incandescent Lamps for Lighting Urban and Suburban Streets.* Harold H. C. Lasker. (8) Jan.
Reversing Electric Motor Drive.* (108) Jan.
The Generation and Transmission of Hydroelectric Power.* E. A. Lof. (9) Jan.
Practical Co-Operation Between Central Station and Isolated Plant. Percival Robert Moses. (9) Jan.
Some Recent Developments in Radiography.* H. Clyde Snook. (3) Jan.
The Use of Motor Drives for Shops. A. G. Popcka. (72) Jan. 2.
The Use of Electric Power in Steel Mills. Stewart C. Coey. (Paper read before the Am. Iron and Steel Inst.) (47) Jan. 3.
Bridging Australia by Wireless.* Alfred Gradenwitz. (26) Jan. 3.
The New Generating Station at the Britannia Works of Messrs. Dorman, Long & Company, Ltd.* (22) Jan. 3.
Notes on Underground Conduit Construction.* Guy F. Speer. (27) Jan. 4.
Arrangement of Exhaust Turbo Generators.* E. Rosenberg. (12) Jan. 10.
Lightning Protection of Buildings. E. J. Berg. (From the *Electrical Review and Western Electrician.*) (73) Jan. 10.
The Properties of Dielectrics in Alternating Current Fields.* (Gutta-Percha.) G. L. Addenbrooke. (73) Jan. 10.
Large Turbo Units.* J. P. Chittenden. (Abstract of paper read before the Rugby Eng. Soc.) (47) Serial beginning Jan. 10.
The Electric Lighting of Villages. W. T. Wardale (26) Jan. 10.

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Electrical—(**Continued**).

Battery-Ringing Telephones.* (From the *English Mechanic* and *World of Science.)* (19) Jan. 11.
New Perth Amboy Central Station.* Warren O. Rogers. (64) Jan. 14.
The Braking of High-Speed Winding Engines.* G. K. Chambers. (Paper read before the South African Institution of Engrs.) (57) Jan. 17.
On the Best Use of a Condenser When Used as a Shunt to a Telephone in Wireless Telegraphy.* H. Smith. (Abstract of a paper read before the Durham Physical Soc.) (73) Jan. 17.
Electrical Apparatus in Iron and Steel and Power Generating Fields; Progress in 1912. By Brent Wiley and H. A. Rapelye. (From the *Electrical Journal.)* (62) Jan. 20.
Performance and Power Consumption of a 2½ cu. yd. Electric Shovel.* C. E. Hogle. (13) Jan. 23.
Standard Sizes of Conduits for Electric Conductors (National Electrical Contractors' Assoc.). (13) Jan. 23.
High-Tension Transmission System in Central Georgia.* (27) Jan. 25.
Temperature and Electrical Insulation.* C. P. Steinmetz and B. G. Lamme. (42) Feb.
Method of Rating Electrical Apparatus. W. L. Merrill, W. H. Powell and Charles Robbins. (42) Feb.
Load Losses of Alternating Current Generators.* W. J. Foster and Edgar Knowlton. (42) Feb.
The Heating of Cables Carrying Current.* Saul Dushman. (42) Feb.
Comparison of Methods of Making Load Tests on A.-C. Generators and on Induction Motors. E. F. Collins and W. E. Holcombe. (42) Feb.
Potential Waves of Alternating-Current Generators.* W. J. Foster. (42) Feb.
Method of Determining Temperature of Alternating-Current Generators and Motors and Room Temperature.* Henry G. Reist and T. S. Eden. (42) Feb.
Laws of Heat Transmission in Electrical Machinery. Irving Langmuir. (42) Feb.
Effect of Room Temperature on Temperature Rise of Motors and Generators.* Maxwell W. Day and R. A. Beekman. (42) Feb.
Notes on Internal Heating of Stator Coils.* R. B. Williamson. (42) Feb.
Methods of Determining Brush Losses Due to Contact and Friction.* H. R. Edgecomb and W. A. Dick. (42) Feb.
A Laboratory Investigation of Temperature Rise as a Function of Atmospheric Conditions.* (As Applied to Electrical Machinery.) C. R. Blanchard and C. T. Anderson. (42) Feb.
Losses in Transformers. W. W. Lewis. (42) Feb.
Methods of Determining Temperature of Transformers.* W. M. McConahey and C. Fortescue. (42) Feb.
Determination of Load Loss Correction Factors for Rotating Electric Machines.* E. M. Olin and S. L. Henderson. (42) Feb.
Effect of Air Temperature, Barometric Pressure and Humidity on the Temperature Rise of Electric Apparatus. C. E. Skinner, L. W. Chubb and Phillips Thomas. (42) Feb.
Determination of Stray Losses from Input-Output Tests.* L. T. Robinson. (42) Feb.
A Proposed Wave Shape Standard.* Cassius M Davis. (42) Feb.
Stray Losses in Induction Motors. A. M. Dudl . (42) Feb.
Methods of Determining Temperature of Tra formers and of Cooling Medium. S. E. Johannesen and G. W. Wade. (42) eb.
Stray Loss in Direct-Current Commutating Ma nes.* H. F. T. Erben and H. S. Page. (42) Feb.
Regulation of Definite Pole Alternators. Sore l. Mortensen. (42) Feb.
The Sphere Spark Gap.* S. W. Farnsworth a C. L. Fortescue. (42) Feb.
Stray Losses in Transformers. C. Fortescue d W. M. McConahey. (42) Feb.
Load Tests on Transformers.* J. J. K. Madden. (42) Feb.
The Temperature Rise of Stationary Induction Apparatus as Influenced by the Effects of Temperature, Barometric Pressure and Humidity of the Cooling Medium.* J. J. Frank and W. O. Dwyer. (42) Feb.
Brush-Friction and Contact-Losses.* H. F. T. Erben and A. H. Freeman. (42) Feb.
Wave Form Distortions and Their Effects on Electrical Apparatus. P. M. Lincoln. (42) Feb.
Current Rating of Electric Cables. Ralph W. Atkinson and H. W. Fisher. (42) Feb.
Sources of Error in Transformer Tests. W. M. McConahey and C. Fortescue. (42) Feb.
Measurement of Temperature in Rotating Electric Machines.* L. W. Chubb, E. I. Chute and O. W. A. Oelting. (42) Feb.
The Experimental Determination of the Regulation of Alternators. A. B. Field. (42) Feb.
Correction on Transformer Temperatures for Variation in Room Temperature, Taking Into Account Both Copper and Iron Losses. C. Fortescue. (42) Feb.

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Electrical—(Continued).

Notes on Methods of Making Load Tests on Large Induction Motors. A. M. Dudley. (42) Feb.
The Calibration of the Sphere Gap Voltmeter.* L. W. Chubb and C. Fortescue. (42) Feb.
Commutation and Brush Loss. C. E. Wilson. (42) Feb.
Sources of Error in the Efficiency Determination of Rotating Electric Machines. Elmer I. Chute and William Bradsaw. (42) Feb.
Comparison of Methods of Loading Large A.-C. and D.-C. Generators and Synchronous Converters for Factory Temperature Tests. F. D. Newbury. (42) Feb.
Les Sécurités Electriques Appliquées aux Installations de Signallations de Signalisation à Manœuvre Manuelle. G. Yseboodt. (30) Dec.
Contribution à l'Etude de la Variation des Propriétés Magnétiques des Fers et Aciers en Fonction de la Température.* Schnieder et Cie. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Jan.
Note sur les Propriétés Magnétiques des Toles pour Dynamos.* De Nolly et Veyret. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Jan.
Beseitigung von Ueberspannungen an Elektromagneten.* P. Kuschewitz. (41) Dec. 26.
Ueber den Bau ruhender Transformatoren.* (41) Dec. 26.
Technische Mitteilung über eine Neukonstrucktion von Luftkondensatoren.* Harald Schering und Rudolf Schmidt. (41) Dec. 26.
Die Betriebssicherheit der Oelschalter.* Max Vogelsang. (41) Jan. 2.
Neuerungen im Bau elektrischer Aufzüge.* W. Feld. (107) Serial beginning Jan. 4.
Neuerungen an selbsttätigen Schaltern für Pumpenanlagen.* (41) Jan. 9.
Ueber die Theorien der Dielektrika.* Tcheslas Bialobjeski. (41) Jan. 9.
Ueber Unterbrechungslichtbogen bei elektrischen Schaltapparaten.* Wilhelm Hoepp. (41) Serial beginning Jan. 9.
Ueber die Berechnung von Pupin-Doppel- und Viererleitungen.* F. Lüschen. (41) Jan. 9.
Die Elektrizitätswerke am Witwatersrand in Transvaal.* A. van der Ham. (41) Jan. 9.
Zur Arbeitsmessung in Drehstromnetzen durch Zähler mit nur einem messenden System.* Karl Schmiedel. (41) Jan. 16.

Marine.

On Shearing Stress in a Ship's Structure.* K. Suyehiro. (Paper read before the Japanese Inst. of Naval Archts.) (11) Dec. 27.
The Generation and Electrical Transmission of Power for Marine Transportation. W. P. Durtnall. (Abstract of paper read before the Soc. of Engrs.) (73) Dec. 27.
Safety of Life at Sea. James Donald. (3) Jan.
Stability of Ships, Notes for Commanders.* George Nicol. (95) Jan.
Shipbuilding and Marine Engineering in 1912.* (11) Jan. 3.
Experimental Tugboats.* (19) Jan. 4.
The French Dreadnoughts *Paris* and *France*.* (12) Jan. 10.
The Starting of Diesel Marine Engines.* (12) Jan. 10.
Lengthening the Aberdeen Liners *Marathon* and *Miltiades*.* (12) Jan. 17.
Development in Marine Condensers. William Weir. (Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (64) Jan. 21.
A New Canadian Oil Engined Ship.* (96) Jan. 23.
Essais Mécaniques des Aciers Spéciaux pour les Constructions Navales.* Leonardo Fea. (Abstract of paper read before the Inter. Assoc. for Testing Materials.) (93) Jan.
Drague Suceuse-Porteuse-Refouleuse Construite par les Ateliers Conrad, de Haarlem.* Jean Guerin. (33) Jan. 18.
Boot aus Eisenbeton.* Hermann Roch. (78) Jan. 3.

Mechanical.

Roller and Ball Bearings.* John Goodman. (63) Vol. 189.
The Economic Use of Lubricating Oils.* David A. Corey. (Abstract of paper read before the National Assoc. of Cotton Mfrs.) (47) Dec.
Calorific Control of Gas-Making: Its Use and Limitations.* Jacques Abady. (66) Dec. 24.
The Electrical Measurement of Wind Velocity.* J. T. Morris. (Paper read before the British Assoc.) (11) Dec. 27.
Messrs. J. Sadd & Sons' Timber Mills at Maldon, Essex.* (26) Dec. 27.
The Lightning Kiln for Calcining Ores or Limestone.* (22) Dec. 27.
The Glover-West Installation of Vertical Retorts.* (66) Dec. 31.
Sewage Sludge for Making Gas. Léonce Fabre. (Paper read before the Société Technique du Gaz.) (66) Dec. 31.
Value of a Weight-Balance in Coal-Gas Manufacture. C. J. Ramsburg. (Paper read before the Am. Gas. Inst.) (66) Dec. 31; (24) Jan. 6; (83) Feb. 1.

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Mechanical (Continued).

Some Thoughts on the Development of the Sand Lime Brick Industry.* P. L. Simpson. (Paper read before the Sand Lime Brick Mfrs. Assoc.) (67) Jan.
Motor Transportation as an Aid to Industrial Economy.* Rollin W. Hutchinson, Jr. (9) Jan.
Steam Meters.* J. A. Knesche. (9) Jan.
Oil Engines Design and Use.* William T. Price. (Paper read before the Philadelphia Foundrymen's Assoc.) (108) Jan.
Foundry Tests and Results. W. R. Dean. (Paper read before the Am. Foundrymen's Assoc.) (108) Jan.
Ceramics Advanced by Tunnel Kiln.* J. A. Seager. (76) Jan. 1.
Sulphur in Illuminating Gas, and Its Removal, with Special Refernce to the Use of Lime. L. J. Willien. (83) Jan. 1.
Great Industrial Establishments, Riter-Conley Manufacturing Co., Pittsburgh, Pa.* (83) Jan. 1.
A Foundry with Continuous Molding Units.* (20) Jan. 2.
Production Efficiency in Typewriter Building.* (20) Jan. 2.
Specifications for Machinery Castings. John Jermain Porter. (20) Jan. 2.
A Complete Milling Machine Test.* P. B. Vernon. (Paper read before the Manchester Assoc. of Engrs.) (20) Jan. 2.
Lime-Sand Bricks.* (Manufacture.) (96) Jan. 2.
A Modern Plant for Automobile Parts.* (20) Jan. 2.
Combining Steam and Gas Power.* (20) Jan. 2.
Largest Precision Testing Machine.* A. H. Emery, Jr. (72) Jan. 2.
Methods of Economizing Heat.* Charles R. Darling. (29) Serial beginning Jan. 3.
Increasing Steam Plant Efficiencies. Edward Ingham. (26) Jan. 3.
Some New Types of Centrifugal and Turbine Pumps.* (57) Jan. 3.
Horizontal Hydraulic Cotton Baling Press.* (12) Jan. 3.
Gas Rate Decision in New Jersey. (Report of the Board of Public Utility Commrs. of New Jersey.) (17) Jan. 4.
Some Notes on Purification of Gas. R. H. Burdick. (24) Serial beginning Jan. 6.
The Prudential Insurance Co. Plant.* Charles H. Bromley. (64) Jan. 7.
The Gas Supply of Munich.* (66) Serial beginning Jan. 7.
The Manufacture of Mantles for Incandescent Gas Lighting. C. Richard Böhm. (66) Serial beginning Jan. 7.
A Gravel Screening and Washing Plant.* (96) Jan. 9.
Manganese Steel for Machinery Parts.* S. R. Stone. (20) Jan. 9.
Use of Compressed Air in Foundry Practice. Arthur F. Murray. (Abstract of paper read before the Am. Foundrymen's Assoc.) (47) Jan. 10.
Drop Forging. Frank W. Trabold. (Paper read before the Soc. of Automobile Engrs.) (19) Jan. 11.
The Auto Truck for Coal Delivery.* Frank C. Perkins. (19) Jan. 11.
Will the Automobile Be Driven by Kerosene? H. A. Morris. (46) Jan. 11.
The Making of a Pneumatic Automobile Tire.* E. R. Hall. (46) Jan. 11.
The Flameless or Convergent Combustion of Gases.* Jean Mennier. (57) Jan. 10.
The Astoria Plant of the New York Consolidated Gas Company.* C. C. Simpson, Jr. (24) Serial beginning Jan. 13.
Technical Gas Analysis.* P. C. Balcon. (Paper read before the Midland Juniors Gas Assoc.) (66) Jan. 14.
Heat Balance in Steam Boilers. Lionel S. Marks. (64) Jan. 14.
Stone Crushing and Screening, Fairmount, Ill.* K. E. Casparis. (13) Jan. 16.
A Very Heavy Motor-Drawn Truck.* (13) Jan. 16.
New Canadian Pacific Coal Handling Plant.* (20) Jan. 16.
Compressed Air as a Foundry Auxiliary.* William H. Armstrong. (Paper read before the Newark Foundrymen's Assoc.) (20) Jan. 16.
Bullard 36-inch Vertical Turret Lathe.* (72) Jan. 16.
Developing the Engine Lathe in an Auto Shop.* Fred H. Colvin. (72) Serial beginning Jan. 16.
Reversing Motors for Machine Tools.* Charles Fair. (72) Jan. 16.
Test of a Surface Condenser, the Effect of a Spray at the Air-Pump Entrance.* Fred Pickford and Gilbert Cook. (11) Jan. 17.
Materials for Motor-Bus Construction.* (11) Jan. 17.
Coking and By-Product Plant at Clifton Colliery. Cumberland.* (22) Jan. 17.
The "Paragon" Internal-Combustion Engine.* (47) Jan. 17.
The Cutting and Generation of Gear Teeth by Modern Gear-Cutting Machinery.* Vincent Gartside. (Paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Jan. 17.
The Etrich Monoplanes.* Stanley Yale Beach. (19) Jan. 18.
New Plant at the Belfast Gas-Works.* (66) Jan. 21.
Acetylene Lighting. C. Hoddle. (Abstract of paper read before the Illuminating Eng. Soc.) (66) Jan. 21.
Petrol Air-Gas Supply. E. Scott-Snell. (Abstract of paper read before the Illuminating Eng. Soc.) (66) Jan. 21.
Recent Developments in Gas-Measuring Apparatus.* Carl C. Thomas. (Paper read before the American Gas Inst.) (66) Jan. 21.

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Mechanical—(Continued).

Performance and Power Consumption of a 2½-cu. yd. Electric Shovel.* C. E Hogle. (13) Jan. 23.
Designs and Materials for Gages.* Lucian L. Haas. (72) Jan. 23.
The Design of Automobile Springs. David Landau and Asher Golden. (72) Jan. 23.
Efficiency of Steel Cable Fastenings. (14) Jan. 25.
A Gas Engine Refrigerating Plant.* (64) Jan. 28.
Independent Air Pump for Condenser.* William Weir. (Paper read before the Institution of Engrs. and Shipbuilders in Scotland.) (64) Jan. 28.
A Method of Proximate Analysis for the Commercial Evaluation of Coal. U. S. Bureau of Mines. (86) Jan. 29.
Workmen's Skill *vs.* Modern Machinery.* Fred. H. Colvin. (72) Jan. 30.
The Design of Hydraulic Intensifiers.* A. Lewis Jenkins. (72) Jan. 30.
Machine Shop Practice of General Interest.* Alexander Taylor. (72) Jan. 30.
A Retail Coal Handling Plant on the Pacific Coast.* (96) Jan. 30.
Air Measurement by Pitot Tubes.* (13) Jan. 30.
A Two-Piece Small Type Converter.* A. F. Blackwood. (20) Jan. 30.
Efficiency of Motor Trucks with Trailers, Report of a Series of Tests Made under Various Conditions at Troy, Ohio.* Morgan Cilley, M. Am. Soc. C. E. (14) Feb. 1.
High Pressure Distribution at Utica.* W. J. Cahill. (83) Feb. 1.
Gasoline and Oil Power on the Farm. Philip S. Rose. (46) Feb. 1.
Economics of the Farm Tractor.* Philip S. Rose. (46) Feb. 1.
Utilisation de la Naphtaline comme Combustible dans les Moteurs à Explosions.* L. Ventou-Duclaux. (32) Oct.
Dispositifs Réduisant la Formation du Poussier, Pendant l'Embarquement du Charbon dans les Navires.* J. E. Giraud. (33) Serial beginning Dec. 7.
Les Véhicules Industriels au Salon de l'Automobile, à Paris, en 1912.* D. Duaner. (33) Dec. 21.
Le Problème de la Turbine à Gaz.* E. Grauce. (37) Dec. 31.
Nouvelle Méthode d'Essai des Soudures.* Ch. Frémont. (Abstract of paper read before the Inter. Assoc. for Testing Materials.) (93) Jan.
Presses à Agglomérer les Charbons, pour la Fabrication des Boulets Ovoïdes. Dupuy frères et Cie. (34) Jan.
Monorail Electrique, Système Abel Pifre, Employé pour la Construction du Grand Collecteur de Nantes.* (34) Jan.
Benzolgewinnung aus Koksofengasen.* W. Friz-Zabrze. (52) Nov. 15.
Die Versuchsanlage für den Wettbewerb um den Kaiserpreis für den besten deutschen Flugzeugmotor.* F. Bendemann. (48) Nov. 16.
Darstellung der Betriebsvorgänge bei Kreiselpumpen.* H. A. Janssen. (48) Nov. 23.
Die Fräsmaschinen der Werkzeugmaschinenfabrik und Eisengiesserei von Droop & Rein in Bielefeld.* F. Nickel. (48) Dec. 7.
Vergleichende Untersuchungen an Wasserstrahl-Luftpumpen.* Grunewald. (48) Serial beginning Dec. 7.
Die spezifische Wärme und das spezifische Volumen des Wasserdampfes für Drücke bis 20 at und Temperaturen bis 550° C.* Max Jakob. (48) Dec. 7.
Kräfteverteilung und Greifen bei Selbstgreifen.* Pfahl. (48) Serial beginning Dec. 14.
Kabelkrane für Bauzwecke.* (40) Dec. 21.
Eine selbsttätige Anlage zur Aufbereitung von Formsand.* G. Geiger. (50) Dec. 26.
Die Wasserstation mit Benoidgasanlage in Pörsten.* Glinski. (102) Jan. 1.
Ueber Umkehrstrassenantriebe.* Georg Meyer. (50) Jan. 2.
National-Kessel für grosse Anlagen.* (7) Jan. 4.
Schüttfeuerungen für Braunkohlen. M. Grellert. (7) Jan. 4.
Ueber die Abhitzeverwertung bei Siemens-Martin-Oefen.* J. Schreiber. (50) Serial beginning Jan. 9.

Metallurgical.

The Fried. Krupp Steel Works, Annen.* (11) Dec. 27.
Investigation of the Iron Melting Problem.* A. W. Belden. (Paper read before the Am. Foundrymen's Assoc.) (47) Dec. 27.
Metallurgy at Tonopah.* M. W. von Bernewitz. (103) Dec. 28.
Progress in Gold-Silver Ore Treatment during 1912.* Alfred James. (103) Dec. 28.
The Electric Furnace in the Production of Iron from Ore.* D. A. Lyon. (105) Jan.
Cyaniding Slimy Ore by Continuous Decantation.* H. C. Parmelee. (105) Jan.
Some Present Pickling Methods. Oliver W. Storey. (105) Jan.
Mammoth System of Fume Control.* Al. H. Martin. (45) Jan.
Progress in the Metallurgy of Iron and Steel. Bradley Stoughton. (20) Jan. 2; (16) Jan. 11.
The Jones & Laughlin Aliquippa Works.* (20) Jan. 2.
Zinc Production and Smelting in 1912. R. G. Hall. (103) Jan. 4.
Progress of Copper Metallurgy.* Thomas T. Read. (103) Jan. 4.

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Metallurgical—(Continued).

Cyaniding at the Wasp No. 2 Mill, Black Hills.* Jesse Simmons. (82) Jan. 4.
Progress in Ore Dressing. Henry S. Munroe. (103) Serial beginning Jan. 4.
Copper Alloys for Motor-Car Service. W. H. Barr. (Abstract of paper read before the Soc. of Automobile Engrs.) (47) Jan. 10.
The Metallurgy of Zinc. W. R. Ingalls. (16) Jan. 11.
Analysis of Cyanide Practice. Herbert A. Megraw. (16) Jan. 11.
Electric Arc Welding on the Pacific Coast.* (17) Jan. 11.
The Theoretical Effect of Increasing the Oxygen of the Blast Supplied to Blast Furnaces. Charles A. Edwards. (Paper read before the Cleveland Institution of Engrs.) (22) Jan. 17.
The Skoda Steel Works, Pilsen, Bohemia.* (12) Jan. 17.
Metallurgy of the Homestake Ore.* (16) Serial beginning Jan. 18.
The New No. 3 Mill at Flat River, Missouri.* S. R. Stone. (82) Jan. 18.
Lead Plant of the International Smelter.* L. S. Austin. (103) Jan. 18.
New Blast Furnace of Maryland Steel Company.* (20) Jan. 23.
Crystalline Growth of Strained Ferrite.* Albert Sauveur. (Paper read before the Inter. Congress for Testing Materials.) (20) Jan. 23.
The Igneous Concentration of Zinc Ores.* F. L. Clerc. (16) Jan. 25.
Electrolysis of Low Grade Gold Bullion. Theodore W. Bouchelle. (16) Jan. 25.
Treatment of Concentrate at the Goldfield Consolidated Mill. J. W. Hutchinson. (103) Serial beginning Jan. 25.
The MacNamara Mill, Tonopah.* M. W. von Bernewitz. (103) Jan. 25.
Smelting Iron by Electricity on the Pacific Coast.* (62) Jan. 27.
Studien über die im Hochofen zwischen den Eisenerzen und Gasen obwaltenden Verhältnisse.* Norbert Metz. (50) Jan. 16.

Military.

Notes on Search-Light Mirrors at the Engineer School.* William F. Endress. (100) Jan.
The Adoption of an Automatic Gun. V. Lelen. (44) Serial beginning Jan.
Military Aviation and Aeronautics. B. D. Foulois. (44) Jan.
Progress in Military Explosives.* Wilford J. Hawkins. (13) Jan. 2.
The Modern Automobile Torpedo.* Robert G. Skerrett. (46) Jan. 4.
New Hangars for Military Uses.* (46) Jan. 18.
La Construction et la Pose des Mines Sous-Marines Système Elia.* (33) Jan. 4.

Mining.

A Plea for the Use of High Tension Constant Continuous Currents in Mines. Sydney F. Walker. (26) Dec. 27.
Electrical Symbols for Mine Maps.* H. H. Clark. (From *Technical Paper 22*, U. S. Bureau of Mines.) (57) Dec. 27.
Graphic Representation of Oilfield Structure.* Alexander J. Heindl. (103) Dec. 28.
Rock House of Quincy Mining Co.* T. C. Desoller. (Paper read before the Lake Superior Min. Inst.) (45) Jan.
Phosphate Ore Dressing.* Strauss L. Lloyd. (45) Jan.
Lake Superior and Cuban Iron Ores.* Day Allen Willey. (45) Jan.
Washery at Soddy Mine, Tennessee.* Frank E. Mueller. (45) Jan.
Mining Steep Dipping Coal.* A. A. Steel. (45) Jan.
Robbing Coal Dust of Its Dangers. Sim Reynolds. (45) Jan.
Coal Mining in India.* J. R. R. Wilson. (Paper read before the National Assoc. of Colliery Mgrs.) (22) Serial beginning Jan. 3.
Coal Washing Plant at a Colliery.* (12) Jan. 3.
Quicksilver Production and Prices.* Clifford G. Dennis. (103) Jan. 4.
Review of Gold-Dredging in 1912.* Charles Janin. (103) Jan. 4.
Mining at the Wasp No. 2, in the Black Hills, South Dakota.* Jesse Simmons. (16) Jan. 4.
Hoisting Practice in the Wisconsin Zinc Fields. W. F. Boericke. (16) Jan. 4.
Electric Power in the Kern and Midway Oil Fields.* Warren Aikens. (82) Jan. 11.
Notes on Diamond Drilling in the Porcupine District, Ont. Albert E. Hall. (From *School of Mines Quarterly.*) (82) Jan. 11.
Difficulties of Pumping on the Comstock Lode. Whitman Symmes. (Abstract of paper read before the California Miners' Assoc.) (82) Jan. 11.
Operations of the Davidson Ore Mining Co., Mich.* Geo. E. Edwards. (82) Jan. 11.
Winding Engine Controllers.* James Black. (Paper read before the National Assoc. of Colliery Mgrs.) (22) Jan. 17.
Shaft-Sinking for the Rondout Siphon.* J. F. Springer. (From *Western Engineering.*) (103) Jan. 18.
Filling of Mine Stopes with Mill Tailings. W. H. Storms. (Paper read before the California Miners' Assoc.) (82) Jan. 18.
Cobalt in 1912. Joseph T. Mandy. (68) Jan. 18.

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Mining—(Continued).

Method and Cost of Constructing a Concrete Lined Shaft by Sinking Through Overburden and Enlarging a Drift Raised Through Ledge Rock.* (86) Jan. 22.
Mining on the West Coast of South America.* Howland Bancroft. (103) Jan. 25.
South Africa. (Mining). Rowland Gascoyne. (82) Jan. 25.
Canada; British Columbia. (Mining). E. Jacobs. (82) Jan. 25.
Installing a Mining Plant in Latin America.* Edward W. Perry. (9) Feb.
Lining a Deep Shaft with Concrete. (14) Feb. 1.
Construction of No. 3 Shaft, Negaunee Mine.* S. R. Elliott. (16) Feb. 1.

Miscellaneous.

Brief Review of Engineering Practice and Personal Experience in Latin America Thirty-Two Years, 1880-1912. Elmer L. Corthell. (4) Dec.
The Electrical Measurement of Wind Velocity.* J. T. Morris. (Paper read before the British Assoc.) (11) Dec. 27.
The Present Opportunities and Consequent Responsibilities of the Engineer. Alexander C. Humphreys. (8) Jan.
Illumination of Interiors.* Preston S. Millar. (Paper read before the National Elec. Light Assoc. and the Illuminating Eng. Soc.) (83) Jan. 1.
Snow Slide Protection at Marble, Colorado.* Homer V. Knouse. (14) Jan. 18.
The Organization of a Corps of Civil Engineers for Public Works Services in the Dominion of Canada. (96) Jan. 23.
Extinguishing Fires with Sawdust. Edwin A. Barrier. (13) Jan. 30.

Municipal.

New Methods of Approaching the Smoke Problem. Osborn Monnelt. (4) Dec.
Delhi, the Metropolis of India.* *Sir* Bradford Leslie. (29) Dec. 27.
Grade and Surface Required on Roads.* (60) Jan.
The Minneapolis Park System.* (60) Jan.
The Kansas City Park System.* James H. Lowry (60) Jan.
Creosoted Block Paving in Chicago.* John Ericson. (60) Jan.
Specifications for Roads of Different Types. E. A. James. (96) Jan. 2.
History of Fifth Avenue Asphalt Pavement, New York. Clifford Richardson, M. Am. Soc. C. E. (14) Jan. 4.
Supplementary Reports on the 1907, 1908, 1909 and 1910 Dust Prevention and Road Preservation Experimental Work of the U. S. Office of Public Roads. (86) Jan. 8.
Methods of Preparing Cement Concrete Pavements. Frank F. Rogers. (Abstract of paper read before the Am. Assoc. for the Advancement of Science.) (86) Jan. 8.
Cost of Object Lesson Sand-Clay Roads Constructed in 1911-12 by the U. S. Office of Public Roads. Logan Waller Page. (86) Jan. 8.
A Method of Preparing Plans and Cross-Sections for Road Grading. (86) Jan. 8.
Distillation of Tar: Methods of Determination and Value in Specifications. Philip P. Sharples. (Paper read before the Am. Assoc. for the Advancement of Science.) (86) Jan. 8.
Reinforced Concrete Elevated Sidewalks, East St. Louis, Ill.* E. R. Rodenberg. (13) Jan. 9.
Why Smoke Is an Industrial Nuisance.* R. C. Benner. (20) Jan. 9.
The Laying Out of Public Parks.* Thomas Mawson. (Report to the Preston Corporation.) (104) Jan. 10.
Europe's Good Roads.* Francis Miltoun. (46) Jan. 11.
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The Traffic Census as a Preliminary to Road Improvement. William D. Sohier. (Abstract of paper read before the Am. Good Roads Congress.) (86) Jan. 22.
The Value of the Traffic Census in the Economical Design of Highways. Arthur H. Blanchard. (Paper read before the Am. Good Roads Congress.) (86) Jan. 22.
King Edward VII Highway. H. S. Van Scoyoc, Assoc. M. Am. Soc. C. E. (96) Jan. 23.
Maintenance and Repair of Asphalt, Bitulithic and Creosoted Wood Block Pavements. W. L. Hempelmann. (Paper read before the Illinois Soc. of Engrs. and Surveyors.) (86) Jan. 29.
Experimental Road Construction at Chevy Chase, Md., by the U. S. Office of Public Roads. (86) Jan. 29.

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Timber for Creosoted Block Paving. Harry C. Davis. (Paper read before the Am. Wood Preservers' Assoc.) (96) Jan. 30.
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Bituminous Gravel Concrete Pavement. Spencer J. Stewart. (Abstract of paper read before the Am. Assoc. for the Advancement of Science.) (14) Feb. 1.
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Teer als Baumaterial für Stadtstrassen. F. Raschig. (39) Dec. 20.
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Essais de Traction à Courant Monophase, à 12 000 Volts, de la Compagnie des Chemins de Fer du Midi.* A. Bidault des Chaumes. (33) Serial beginning Dec. 28.
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Recherches Américaines sur les Rails Poursuivies Conjointement par les Chemins de Fer et les Aciers.* H. Wickhorst. (Abstract of paper read before the Inter. Assoc. for Testing Materials.) (93) Jan.
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Some Points in the Design and Construction of Reinforced Concrete. E. P. Wells. (Abstract of paper read before the Concrete Inst.) (13) Jan. 2.
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Nouvelle Méthode d'Essai des Tuyaux en Acier.* Ch. Frémont. (Paper read before the Inter. Assoc. for Testing Materials.) (93) Jan.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XXXIX. FEBRUARY, 1913. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

EXPERIMENTS ON WEIR DISCHARGE.*

By W. G. Steward,† Esq., and J. S. Longwell,‡ Jun. Am. Soc. C. E.

The experiments discussed in this paper were made by the writers at the experiment station of the Boise Project of the United States Reclamation Service during 1911 in connection with duty of water investigations for the project. The project experiment station is on the main canal of the Boise Project, near Boise River, at a point about 8 miles above Boise, Idaho, and was built in the fall of 1910.

The water used for this station is diverted into a forebay from the main canal by a double turn-out consisting of two 18-in. concrete pipes, each controlled separately by gates on the bank of the main canal, as shown by Fig. 1. The forebay or weir pool is 30 ft. square on the bottom, 4 ft. deep, and has side slopes of 1½ to 1. The bottom of this bay is 12 ft. below the bottom of the main canal. An adjustable baffle-board divides the forebay into two parts. The sides of the pool above the baffle are rip-rapped with large stones in order to prevent the water from washing the banks as it issues from the turn-out. Below the baffle the water is quiet, and no rip-rap is needed.

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

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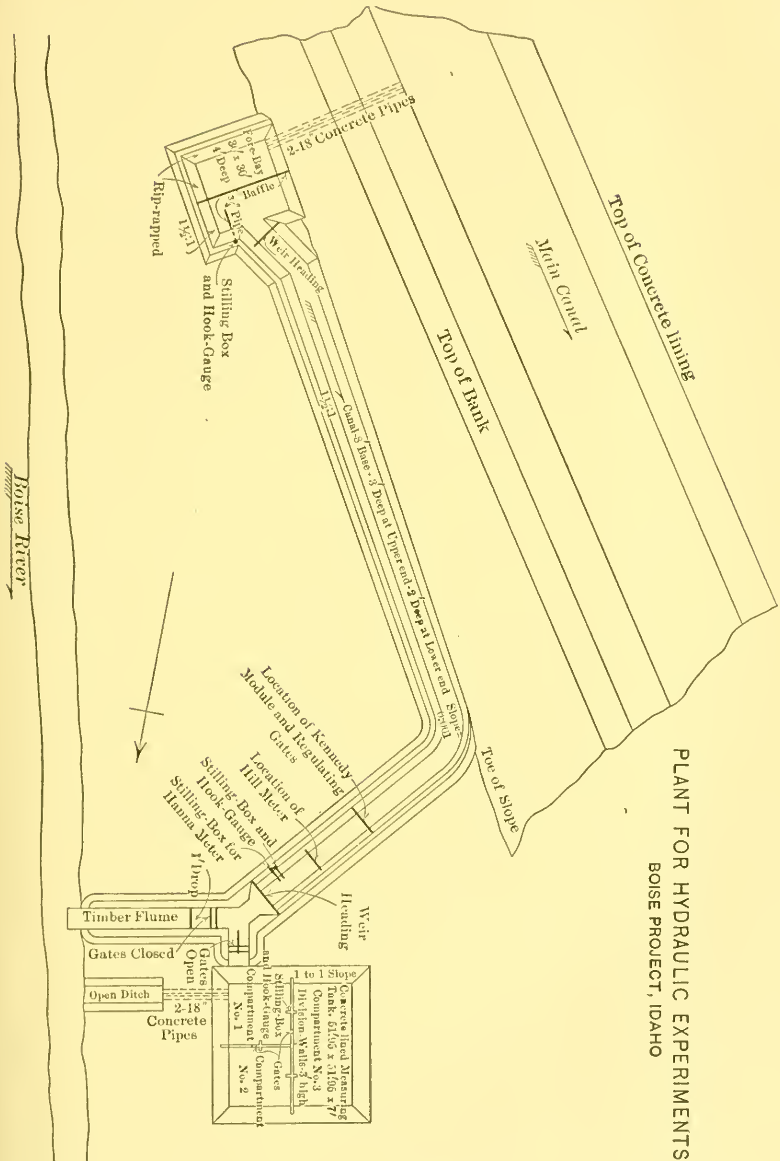
A canal having a bottom width of 8 ft. carries the water about 318 ft. from this forebay to the upper end of a timber flume, 8 ft. wide and 3 ft. deep. This flume section extends for 12 ft. and then divides into two sections of equal width at right angles with each other and located so that the center line of each makes an angle of 45° with the center line of the main flume. The left branch leads to a concrete measuring tank; the right branch leads to Boise River, and acts as a wasteway. A set of gates in each branch controls the flow of water, diverting it into the tank or into the river, as may be desired. There are two gates in each set, a large one hinged to a vertical post set just enough off center to cause the water pressure to keep it closed, and a small water-tight one hinged to the floor and held in a vertical position by a hinged brace fastened to one of the cross-pieces on top of the flume. A water-proof canvas is nailed to the sides and floor of the flume and to this gate, with enough extra material in the ends, so that the gate can be operated without difficulty. The large gate serves to stop most of the flow, thus making it easier to operate the small one. The closing of one set of gates requires from 1 to 2 sec. There was no difficulty in operating the gates, and they proved very satisfactory and efficient.

This gate heading was caulked and coated with a tar preparation, and no leakage could be detected, either through the flume or the gates. The upper portion of the flume, between the gates and the weir, acted as a storage basin for the water between the time of closing one set and the opening of the other. The weir was set high enough above the flume floor so that the water which backed up between the shutting and opening of the gates did not cause any submergence on the weirs. In the case where the weirs were set at the upper heading, the flume and canal acted as the storage basin, and no effect on the weir discharge resulted.

In obtaining the intervals for the various runs, the time was recorded at the closing of the gates in the wasteway at the beginning of the run and the closing of the opposite set at the end of the experiment. Thus the only possible error that might enter into the results would be the difference in times required for closing, which at the most was about ½ sec.

The concrete measuring tank is set into the ground 7 ft. The



PLANT FOR HYDRAULIC EXPERIMENTS
BOISE PROJECT, IDAHO

FIG. 1.

bottom is 51.95 ft. square, and the side slopes are on a batter of 1 to 1. The walls and the floor are 6 in. thick, and have a ½-in. finishing coat which is troweled very smoothly. The walls are in perfect alignment, and present no uneven surfaces of any sort. To facilitate measurements, the lower part of the tank is divided into three compartments by 6-in. walls, 3 ft. high. These are connected by gates which are operated from a walk above the tank. The discharge from the tank into the river is made through two 18-in. pipes which are controlled by gates set in the tank wall. The tank was calibrated accurately by measurements, and had a capacity of 24 000 cu. ft. A number of tests were made for leakage from the tank, but no appreciable quantity could be detected.

A weir heading was constructed at the head of the flume and was used for the 6-in. and 1-ft. weirs. A second weir heading was set at the point where the canal takes out of the forebay. This heading was used for the 2-ft. and 3-ft. weirs, and was of such a size as to permit a 6-ft. weir to be set on it. Both these headings were caulked with oakum and tarred, so that no leakage whatever occurred.

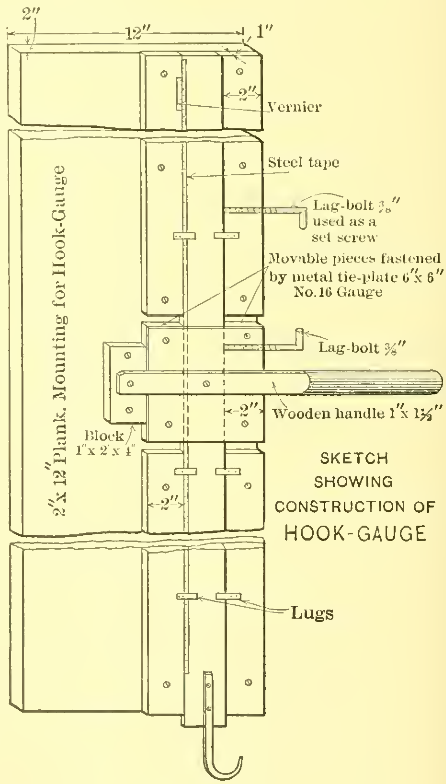


FIG. 2.

Hook-gauges were set above each weir heading, and a third one in the measuring tank.

In the case of the upper heading at the forebay, a stilling-box, 1 ft. square, was set back into the bank, with a ¾-in. pipe leading out into the forebay about 15 ft. above the weir. The hook-gauge was

attached to the inside of the stilling-box, and the head on the weirs was observed above the effect of the velocity of approach.

The stilling-box for the lower weir at the flume heading was of the same size as above; it was set back in the bank of the canal, 10 ft. above the weir, and was connected with the canal by a ¾-in. pipe. The hook-gauge was set in the manner just described.

The hook-gauge used in the measuring tank was placed in a stilling-box, 1 ft. square, fastened to one of the division wall buttresses in Compartment No. 1 and operated from the plank walk.

These hook-gauges, Fig. 2, consisted of a 1 by 2-in. strip with a hook fastened to the lower end, sliding between two similar 1 by 2-in. strips. A length of steel tape, reading to hundredths of a foot, was tacked on this sliding strip. A vernier was screwed on a small block, and the block was then nailed to one of the strips so that the vernier edge almost touched the steel tape on the sliding strip. This enabled the observer to read accurately to thousandths of a foot. In order to hold the slide in place, a set-screw was attached to one of the strips, and, by turning it, the slide was forced into close contact with the opposite strip and held firm. At a position on the gauge about 3 ft. above the ground or walk on which the observer stood, the two nailed strips were cut away for a distance of 6 in., and in their place two strips, considerably shorter than those removed, and fastened together by a metal plate, were inserted. A lever was fastened to a block nailed to the plank holding the gauge at a point opposite the center of the opening, and was also fastened to the metal of the small double slide with a screw. A set-screw was attached to the outer strip of the double slide. Thus, by turning this set-screw against the single slide holding the steel tape and forcing it against the opposite strip of the double slide, the single slide could be moved up and down by the lever, the hook could be brought to the proper position, and the slide was then clamped by the first set-screw. The reading could then be taken. This lever enabled the observer to set the gauge very quickly and also very accurately. These gauges were made by the writers, and served their purpose very effectively.

The weir experiments consisted of calibrations of 6-in., 1-ft., 2-ft., and 3-ft. sharp-crested Cippoletti and rectangular weirs with full contraction. The experiments on sharp-crested Cippoletti and rectangular weirs were made, not with the idea of obtaining new general formulas

for weirs, but for the purpose of testing the general weir formulas and ascertaining whether they are applicable to the weirs on the Boise and other projects of the Reclamation Service, and, if necessary, of preparing rating tables for the weirs tested and a formula or table by which accurate discharges might be obtained for a range of weirs and heads above the limits of previous experiments.

The crests of all the weirs were set about 1 ft. above the canal bottom. The 6-in. and 1-ft. weirs were located in the center of the 8-ft. canal at the flume heading. The 2-ft. and 3-ft. weirs were located at the head of the canal, and took the water directly from the 30-ft. forebay. The gauges were set far enough above the weirs to be free from the effect of velocity of approach. At the flume heading, baffles were placed in the canal just below a regulating gate, and planks were laid on the water to do away with any surface disturbances. A very even and uniform flow resulted, with no appreciable velocity of approach except in one or two high heads on the 1-ft. Cippoletti weir. In the case of the weirs used on the upper heading, the regulation was obtained with the gates in the main canal. Very little trouble was experienced after the water had settled in the pool. The water in the main canal was from 4 to 5 ft. in depth over the gates, and any slight variation in head did not affect the discharge of the pipes appreciably. The hook-gauges above the weirs were read every 30 sec. throughout the length of runs, which varied from 5 min. to 1 hour, depending on the head. The average of the readings gave the gauge height for each particular run. The different compartments of the tank were used as the discharge increased. For the low heads and smaller weirs Compartment No. 1 was used. As the head increased, Compartments Nos. 1 and 2 were used, and, for the large heads, the entire tank was utilized. No trouble was experienced in reading the gauges to thousandths of a foot.

Cippoletti Weirs.—The experiments on Cippoletti weirs are divided into five parts: (1) calibrations of the 6-in. weir; (2) an attempt to determine the shape of the 6-in. weir to discharge according to the Cippoletti formula; (3) calibrations of the 1-ft. weir; (4) calibrations of the 2-ft. weir; and (5) calibrations of the 3-ft. weir.

In the 6-in. weir, twenty-four runs were made with heads ranging from 0.161 to 0.945 ft. The maximum variation in gauge height during any run was 0.024 ft. No appreciable velocity of approach

TABLE 1.—Results of Experiments on Weirs.

Cippoletti Weirs.

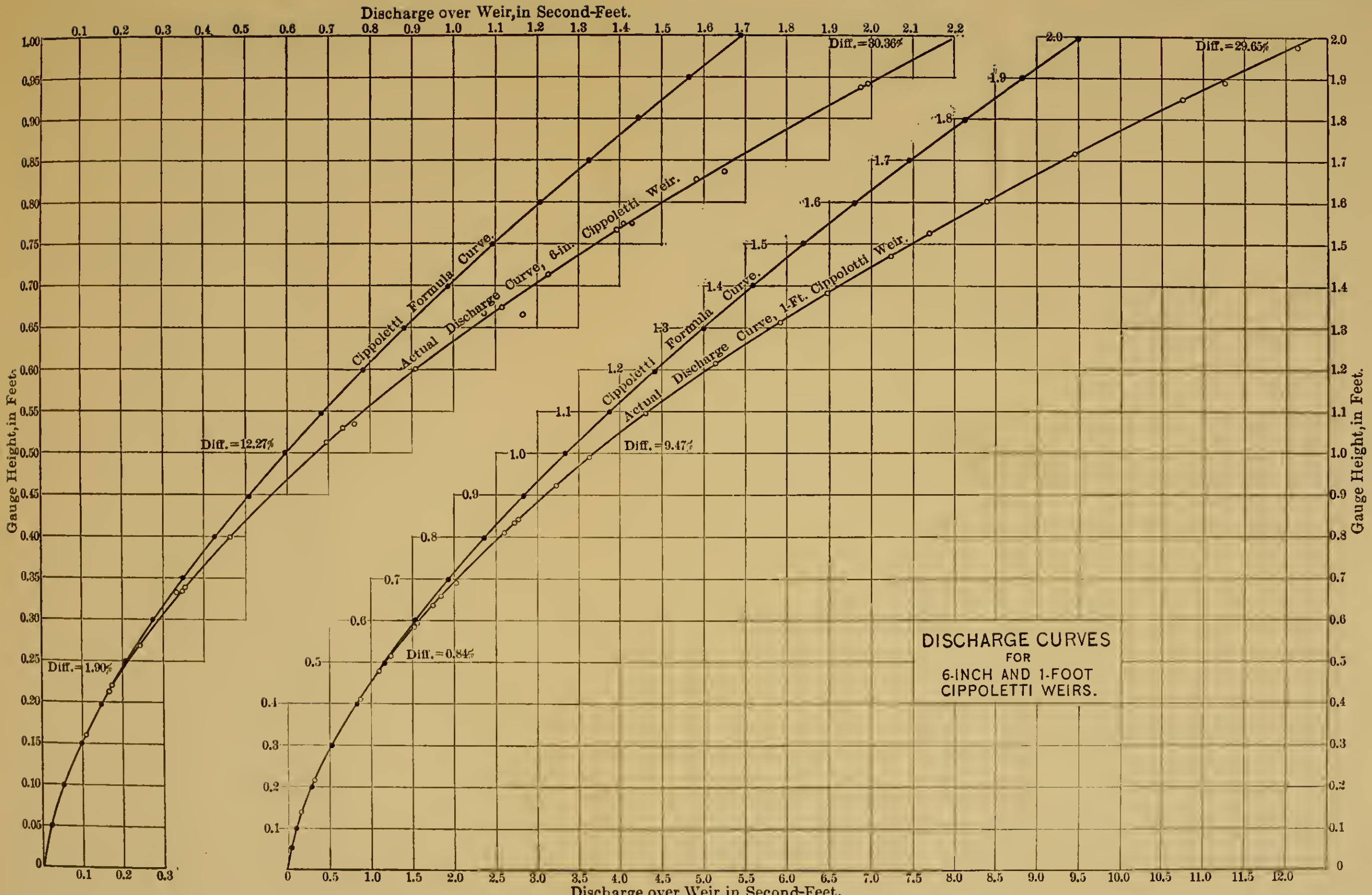
Series No.	Run No.	Average gauge height.	Variation in gauge height.	Length of weir, in inches.	Time interval, in seconds.	Actual discharge.	Discharge, in second-feet.	C in formula $Q = Clh^{\frac{3}{2}}$
		h		l			Q	
1	1	0.217	0.004	6	4 560	781	0.171	3.383
	2	0.225	0.000	6	5 100	913	0.179	3.352
	3	0.334	0.024	6	2 970	978	0.329	3.408
	4	0.336	0.002	6	3 935	1 365	0.347	3.562
	5	0.334	0.002	6	2 490	855	0.343	3.552
	6	0.513	0.004	6	2 620	1 822	0.695	3.783
	7	0.667	0.001	6	1 625	1 894	1.166	4.284
	8	0.770	0.005	6	1 365	1 905	1.396	4.132
	9	0.941	0.000	6	880	1 739	1.976	4.329
	10	0.839	0.012	6	1 075	1 773	1.649	4.291
	11	0.776	0.017	6	1 255	1 788	1.424	4.166
	12	0.675	0.000	6	1 255	1 402	1.117	4.028
	13	0.537	0.000	6	1 392	1 059	0.761	3.867
	14	0.940	0.009	6	970	1 918	1.977	4.338
	15	0.945	0.005	6	900	1 793	1.992	4.337
	16	0.774	0.001	6	1 380	1 939	1.405	4.126
	17	0.715	0.000	6	1 165	1 428	1.226	4.055
	18	0.600	0.000	6	945	856	0.906	3.898
	19	0.400	0.000	6	990	460	0.465	3.676
	20	0.830	0.000	6	1 200	1 897	1.581	4.181
	21	0.667	0.000	6	1 200	1 292	1.077	3.954
	22	0.530	0.000	6	1 110	813	0.732	3.795
	23	0.279	0.000	6	1 260	307	0.244	3.478
	24	0.161	0.004	6	780	83	0.106	3.287
7	1	0.637	0.007	12	600	1 064	1.773	3.487
	2	0.594	0.010	12	480	759	1.581	3.454
	3	0.588	0.005	12	480	744	1.550	3.437
	4	0.518	0.001	12	480	609	1.269	3.404
	5	0.482	0.002	12	600	687	1.145	3.422
	6	0.409	0.002	12	600	533	0.888	3.394
	7	0.302	0.001	12	600	335	0.558	3.361
	8	0.215	0.002	12	1 200	397	0.331	3.320
	9	0.138	0.001	12	1 200	205	0.171	3.333
	10	0.840	0.019	12	510	1 413	2.771	3.599
	11	0.846	0.002	12	600	1 683	2.805	3.604
	12	0.659	0.006	12	540	1 017	1.883	3.520
	13	0.693	0.002	12	600	1 222	2.037	3.530
	14	0.815	0.004	12	600	1 587	2.645	3.595
	15	0.927	0.004	12	600	1 958	3.263	3.656
	16	0.992	0.003	12	600	2 189	3.648	3.692
	17	1.097	0.003	12	600	2 591	4.318	3.758
	18	1.219	0.003	12	600	3 097	5.162	3.835
	19	1.320	0.003	12	480	2 849	5.935	3.913
	20	1.386	0.002	12	360	2 333	6.481	3.971
	21	1.476	0.004	12	360	2 604	7.233	4.034
	22	.....	.....	..	...	.....	.....	.....
	23	1.528	0.001	12	360	2 771	7.697	4.076
	24	1.604	0.000	12	360	3 014	8.372	4.121
	25	1.721	0.008	12	300	2 836	9.453	4.186
	26	1.853	0.008	12	300	3 227	10.757	4.265
	27	1.897	0.002	12	270	3 042	11.267	4.312
	28	1.978	0.009	12	240	2 915	12.145	4.366
8	1	0.066	0.003	24	1 200	146	0.122	3.588
	2	0.186	0.002	24	1 200	662	0.552	3.441
	3	0.317	0.000	24	600	727	1.212	3.392
	4	0.318	0.002	24	600	727	1.212	3.380
	5	0.430	0.002	24	600	1 146	1.910	3.386
	6	0.438	0.003	24	600	1 172	1.953	3.368
	7	0.526	0.001	24	600	1 550	2.583	3.385
	8	0.521	0.001	24	600	1 541	2.568	3.385
	9	0.601	0.002	24	600	1 890	3.150	3.380

TABLE 1.—(*Continued.*)

Series No.	Run No.	Average gauge height.	Variation in gauge height.	Length of weir, in inches.	Time interval, in seconds.	Actual discharge.	Discharge, in second-feet.	C in formula $Q = Clh^{\frac{3}{2}}$.
		h		l			Q	
8	10	0.594	0.004	24	600	1 859	3.098	3.384
	11	0.588	0.001	24	660	1 824	3.040	3.371
	12	0.668	0.003	24	600	2 235	3.725	3.411
	13	0.679	0.003	24	600	2 286	3.810	3.197
	14	0.853	0.004	24	600	3 257	5.428	3.445
	15	0.854	0.004	24	600	3 266	5.413	3.448
	16	1.086	0.005	24	420	3 227	7.921	3.499
	17	1.090	0.003	24	420	3 359	7.998	3.514
	18	1.238	0.006	24	330	3 234	9.800	3.557
	19	1.484	0.011	24	600	7 871	13.118	3.628
	20	1.101	0.016	24	600	4 844	8.073	3.494
	21	1.425	0.010	24	600	7 350	12.250	3.601
	22	1.537	0.006	24	600	8 247	13·745	3.607
	23	1.598	0.007	24	630	9 391	14.906	3.689
	24	1.816	0.017	24	540	9 822	18.189	3.719
	25	1.852	0.016	24	480	9 004	18.758	3.722
	26	1.987	0.010	24	660	13 981	21.092	3.737
	27	0.406	0.004	24	630	1 108	1.727	3.338
	28	0.290	0.003	24	900	949	1.054	3.374
	29	0.178	0.001	24	900	461	0.512	3.409
	30	0.154	0.003	24	1 200	502	0.418	3.455
	31	0.092	0.002	24	1 800	347	0.193	3.459
	32	0.059	0.002	24	1 800	201	0.112	3.889
	33	0.034	0.001	24	3 660	173	0.047	3.730
11	1	0.172	0.002	36	1 200	883	0.736	3.436
	2	0.170	0.002	36	1 200	867	0.722	3.433
	3	0.082	0.004	36	3 600	909	0.252	3.574
	4	0.236	0.002	36	900	1 068	1.187	3.450
	5	0.302	0.000	36	600	1 025	1.708	3.430
	6	0.609	0.004	36	600	2 920	4.867	3.414
	7	0.500	0.001	36	600	2 164	3.607	3.400
	8	0.460	0.006	36	480	1 521	3.169	3.386
	9	0.297	0.002	36	600	989	1.648	3.393
	10	0.130	0.002	36	900	438	0.487	3.461
	11	0.090	0.001	36	1 260	354	0.281	3.469
	12	0.216	0.001	36	900	924	1.027	3.410
	13	0.414	0.002	36	600	1 793	2.988	3.366
	14	0.598	0.003	36	600	2 831	4.718	3.400
	15	0.844	0.005	36	600	4 764	7.940	3.413
	16	0.994	0.003	36	600	6 146	10.243	3.445
	17	1.167	0.005	36	600	7 796	12.993	3.435
	18	1.410	0.007	36	360	6 262	17.394	3.463
	19	0.396	0.002	36	600	1 506	2.510	3.357
	20	1.470	0.005	36	420	7 836	18.657	3.489
	21	1.923	0.006	36	300	8 404	28.013	3.502
	22	.....	.....	..	...	.....	......	.....
	23	2.415	0.009	36	300	12 416	41.387	3.676
	24	2.246	0.008	36	240	8 560	35.667	3.532
	25	2.065	0.008	36	300	9 418	31.393	3.526

existed during this experiment, and the end contractions and the shape of the nappe were not disturbed in any manner. Compartment No. 1 of the measuring tank was used entirely during the calibrations of this weir.

Plate XXIII shows the gauge height and discharge curve and Table 2 gives the actual discharge for each $\frac{1}{100}$ ft. of gauge height. Plate XXV is a logarithmic discharge curve from which it will be



DISCHARGE CURVES
FOR
6-INCH AND 1-FOOT
CIPPOLETTI WEIRS.

seen that three distinct parabolas exist as the discharge curve, instead of one. This logarithmic curve was used in determining the proper plotting of the gauge height discharge curve. The equations of the different parabolas were determined as shown in Table 7, also the gauge height at which the change of equation takes place, and the proportional part that this gauge height is of the crest length.

TABLE 2.—GAUGE HEIGHT AND DISCHARGE FOR 6-INCH CIPPOLETTI WEIR.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.00			0.51		0.691	
0.01		0.002		0.52		0.713	
0.02		0.005		0.53		0.735	
0.03		0.009		0.54		0.758	
0.04		0.013		0.55		0.781	
0.05	0.019	0.019		0.56		0.805	
0.06		0.025c		0.57		0.830	
0.07		0.031		0.58		0.855	
0.08		0.038		0.59		0.880	
0.09		0.045		0.60	0.782	0.906	15.86
0.10	0.053	0.053		0.61		0.932	
0.11		0.061		0.62		0.959	
0.12		0.070		0.63		0.987	
0.13		0.079		0.64		1.015	
0.14		0.088		0.65		1.043	
0.15		0.098		0.66		1.071	
0.16		0.108		0.67		1.099	
0.17		0.118		0.68		1.128	
0.18		0.129		0.69		1.157	
0.19		0.139		0.70	0.986	1.186	20.28
0.20	0.151	0.151		0.71		1.215	
0.21		0.162		0.72		1.245	
0.22		0.174		0.73		1.275	
0.23		0.186		0.74		1.305	
0.24		0.200		0.75		1.336	
0.25		0.214	1.90	0.76		1.367	
0.26		0.228		0.77		1.398	
0.27		0.243		0.78		1.429	
0.28		0.258		0.79		1.460	
0.29		0.273		0.80	1.205	1.492	23.82
0.30	0.277	0.288	3.97	0.81		1.524	
0.31		0.304		0.82		1.557	
0.32		0.320		0.83		1.590	
0.33		0.337		0.84		1.623	
0.34		0.354		0.85		1.657	
0.35		0.371		0.86		1.691	
0.36		0.389		0.87		1.726	
0.37		0.407		0.88		1.761	
0.38		0.425		0.89		1.796	
0.39		0.444		0.90	1.437	1.832	27.49
0.40	0.426	0.463	8.69	0.91		1.868	
0.41		0.482		0.92		1.904	
0.42		0.502		0.93		1.940	
0.43		0.522		0.94		1.976	
0.44		0.542		0.95		2.012	
0.45	0.508	0.563		0.96		2.048	
0.46		0.584		0.97		2.084	
0.47		0.605		0.98		2.120	
0.48		0.626		0.99		2.157	
0.49		0.647		1.00	1.683	2.194	30.36
0.50	0.595	0.669	12.45				

A number of runs were made to determine the shape of a weir that would discharge according to the Cippoletti formula. Plates were screwed on the face of a 6-in. Cippoletti weir, and a general shape was determined. The shape obtained, Fig. 3, was a series of reverse curves or waves, and seems to be of no practical value.

The 1-ft. Cippoletti weir was placed on the flume heading, and the flow was regulated by special gates. The gauge heights on this weir ranged from 0.138 to 1.978 ft., with a maximum variation of gauge height of 0.019 ft. for any run. No corrections were made for velocity of approach, as the maximum correction was only 0.002 ft. for the highest head of 1.978 ft. over the weir. The effect on the lower heads was not appreciable. In all cases there was full contraction and a free nappe. A gauge height discharge curve, also shown on Plate XXIII, was plotted with the assistance of a logarithmic curve, as described for the 6-in. weir. Table 3 gives the discharge for each $\frac{1}{100}$ ft. of gauge height. Compartment No. 1 in the tank was used for the lower heads, and Nos. 1 and 2 for the higher ones.

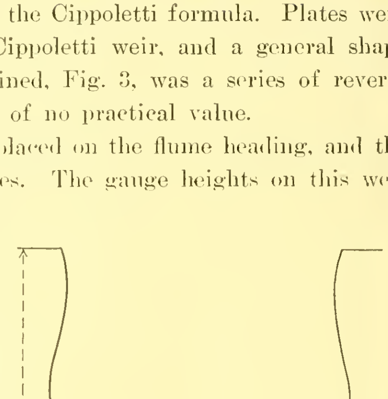


Fig. 3.

The 2-ft. and 3-ft. Cippoletti weirs were set at the head of the canal, taking the water from the 30-ft. forebay. No velocity of approach existed, as the gauge was set to give the elevation of the water in the forebay above the effect of the weirs. For the 2-ft. weirs, the heads ranged from 0.034 to 1.987 ft., with a maximum variation of head of 0.017 ft. for any run. For the 3-ft. weir, the heads varied from 0.082 to 2.415 ft. The greatest variation of head was 0.009 ft. The contraction in both weirs was entirely full, and the nappes were perfectly free from any disturbances. Compartment No. 1 of the measuring tank was used for the low heads, Nos. 1 and 2 for interme-

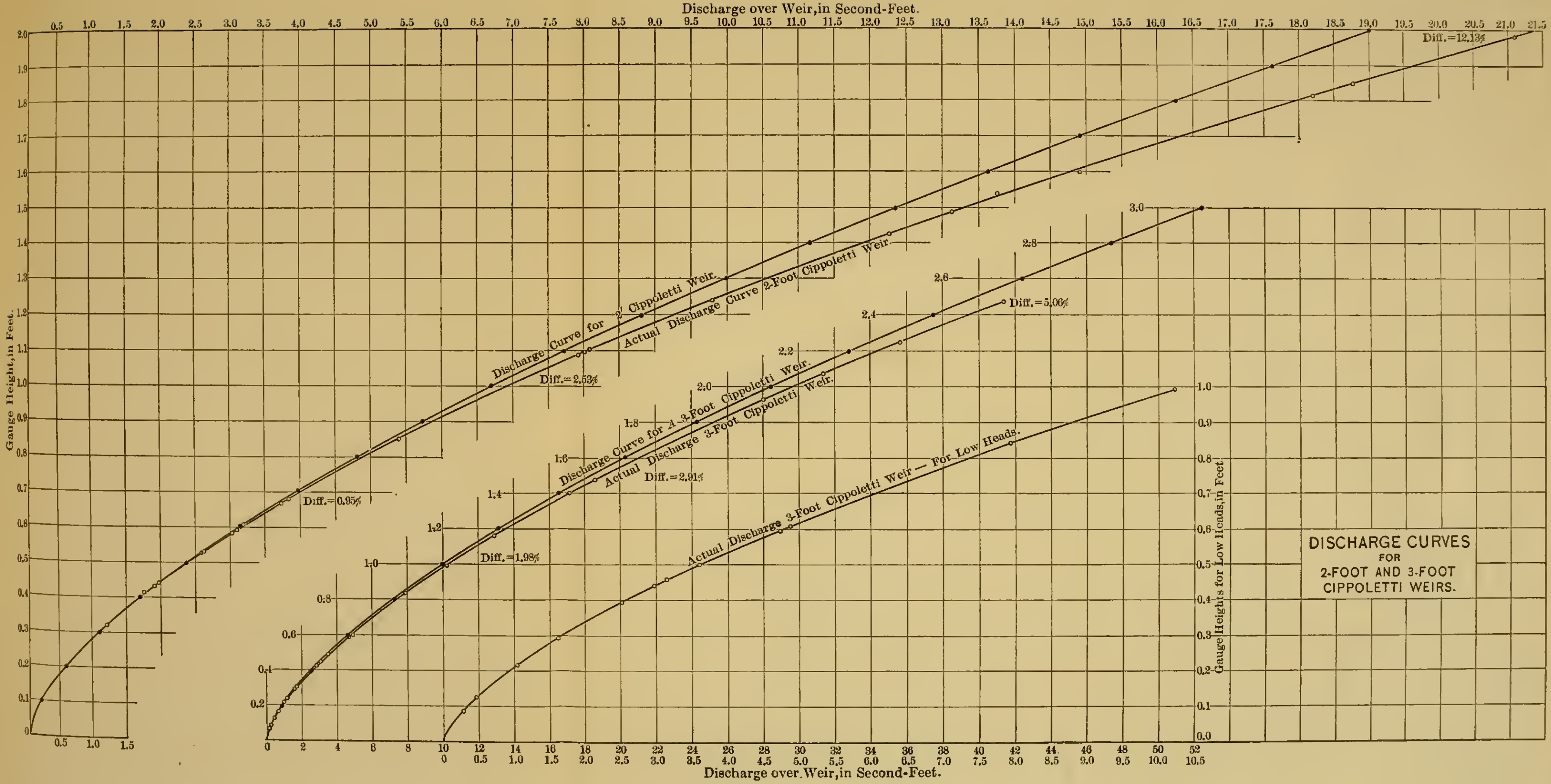

Discharge over Weir, in Second-Feet.
Gauge Height, in Feet.
Discharge Curve for 2' Cippoletti Weir.
Actual Discharge Curve 2-Foot Cippoletti Weir.
Diff.=12.13%
Diff.=2.53%
Diff.=0.95%
Discharge Curve for A 3-Foot Cippoletti Weir.
Actual Discharge 3-Foot Cippoletti Weir.
Diff.=5.06%
Diff.=2.91%
Diff.=1.98%
Actual Discharge 3-Foot Cippoletti Weir — For Low Heads.
Gauge Heights for Low Heads, in Feet.
Discharge over Weir, in Second-Feet.
DISCHARGE CURVES
FOR
2-FOOT AND 3-FOOT
CIPPOLETTI WEIRS.

TABLE 3.—GAUGE HEIGHT AND DISCHARGE FOR 1-FOOT CIPPOLETTI WEIR.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.000	0.000		0.66		1.871	
0.01		0.003		0.67		1.919	
0.02		0.010		0.68		1.968	
0.03		0.018		0.69		2.017	
0.04		0.027		0.70	1.972	2.066	4.77
0.05	0.038	0.038		0.71		2.115	
0.06		0.050		0.72		2.164	
0.07		0.062		0.73		2.213	
0.08		0.076		0.74		2.262	
0.09		0.091		0.75		2.312	
0.10	0.107	0.107		0.76		2.362	
0.11		0.123		0.77		2.412	
0.12		0.140		0.78		2.462	
0.13		0.158		0.79		2.513	
0.14		0.176		0.80	2.409	2.565	6.48
0.15		0.196		0.81		2.618	
0.16		0.216		0.82		2.671	
0.17		0.236		0.83		2.724	
0.18		0.257		0.84		2.777	
0.19		0.279		0.85		2.830	
0.20	0.301	0.301		0.86		2.884	
0.21		0.324		0.87		2.939	
0.22		0.347		0.88		2.994	
0.23		0.371		0.89		3.049	
0.24		0.396		0.90	2.875	3.105	8.00
0.25		0.421		0.91		3.161	
0.26		0.446		0.92		3.217	
0.27		0.472		0.93		3.274	
0.28		0.499		0.94		3.331	
0.29		0.526		0.95		3.389	
0.30	0.553	0.553		0.96		3.447	
0.31		0.581		0.97		3.506	
0.32		0.609		0.98		3.565	
0.33		0.638		0.99		3.625	
0.34		0.667		1.00	3.367	3.686	9.47
0.35		0.697		1.01		3.747	
0.36		0.727		1.02		3.809	
0.37		0.758		1.03		3.872	
0.38		0.789		1.04		3.935	
0.39		0.820		1.05		3.999	
0.40	0.852	0.852		1.06		4.063	
0.41		0.884		1.07		4.127	
0.42		0.916		1.08		4.192	
0.43		0.949		1.09		4.258	
0.44		0.983		1.10	3.884	4.325	11.35
0.45		1.016		1.11		4.393	
0.46		1.050		1.12		4.461	
0.47		1.085		1.13		4.530	
0.48		1.122		1.14		4.600	
0.49		1.161		1.15		4.670	
0.50	1.190	1.200	0.84	1.16		4.740	
0.51		1.239		1.17		4.811	
0.52		1.278		1.18		4.883	
0.53		1.317		1.19		4.955	
0.54		1.356		1.20	4.426	5.027	13.58
0.55		1.396		1.21		5.100	
0.56		1.437		1.22		5.173	
0.57		1.478		1.23		5.247	
0.58		1.519		1.24		5.321	
0.59		1.561		1.25		5.395	
0.60	1.565	1.603	2.43	1.26		5.470	
0.61		1.646		1.27		5.547	
0.62		1.689		1.28		5.624	
0.63		1.732		1.29		5.702	
0.64		1.778		1.30	4.990	5.781	15.85
0.65		1.824		1.31		5.861	

TABLE 3.—(*Continued.*)

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
1.32		5.941		1.67		8.994	
1.33		6.021		1.68		9.088	
1.34		6.102		1.69		9.182	
1.35		6.183		1.70	7.462	9.276	24.31
1.36		6.265		1.71		9.370	
1.37		6.347		1.72		9.464	
1.38		6.429		1.73		9.558	
1.39		6.512		1.74		9.652	
1.40	5.577	6.595	18.25	1.75		9.746	
1.41		6.678		1.76		9.841	
1.42		6.762		1.77		9.937	
1.43		6.846		1.78		10.034	
1.44		6.930		1.79		10.131	
1.45		7.015		1.80	8.130	10.229	25.82
1.46		7.101		1.81		10.327	
1.47		7.187		1.82		10.425	
1.48		7.274		1.83		10.523	
1.49		7.362		1.84		10.624	
1.50	6.185	7.450	20.45	1.85		10.728	
1.51		7.538		1.86		10.833	
1.52		7.626		1.87		10.938	
1.53		7.714		1.88		11.044	
1.54		7.802		1.89		11.150	
1.55		7.890		1.90	8.817	11.256	27.66
1.56		7.979		1.91		11.363	
1.57		8.069		1.92		11.470	
1.58		8.160		1.93		11.577	
1.59		8.251		1.94		11.684	
1.60	6.814	8.343	22.44	1.95		11.791	
1.61		8.435		1.96		11.900	
1.62		8.527		1.97		12.010	
1.63		8.620		1.98		12.120	
1.64		8.713		1.99		12.232	
1.65		8.806		2.00	9.522	12.345	29.65
1.66		8.900					

diate heads, and the entire tank for the high heads. The discharge curves are shown on Plates XXIV and XXV, and Tables 4 and 5 give the details.

Plates XXIII and XXIV show the discharge curves for the actual discharge of the Cippoletti weirs and the theoretical discharge by the Cippoletti formula. It is observed that in every case the actual discharge curve follows the Cippoletti curve to a gauge height of approximately one-third of the length of the crest. After this point, the actual discharges for the gauge height become larger than indicated by the formula. Tables 2 to 5 give the actual discharge for each $\frac{1}{100}$ ft. of gauge height, the formula discharge for each $\frac{1}{10}$ ft. of gauge height, and the percentage that the actual discharge is in excess of the formula discharge for each $\frac{1}{10}$ ft. of gauge height. At a gauge height of 1 ft. on a 6-in. weir, the difference is 30.36%, at 2 ft. on a 1-ft. weir it is 29.65%, showing that at a gauge height equal to twice the length

PLATE XXV.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1913.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.

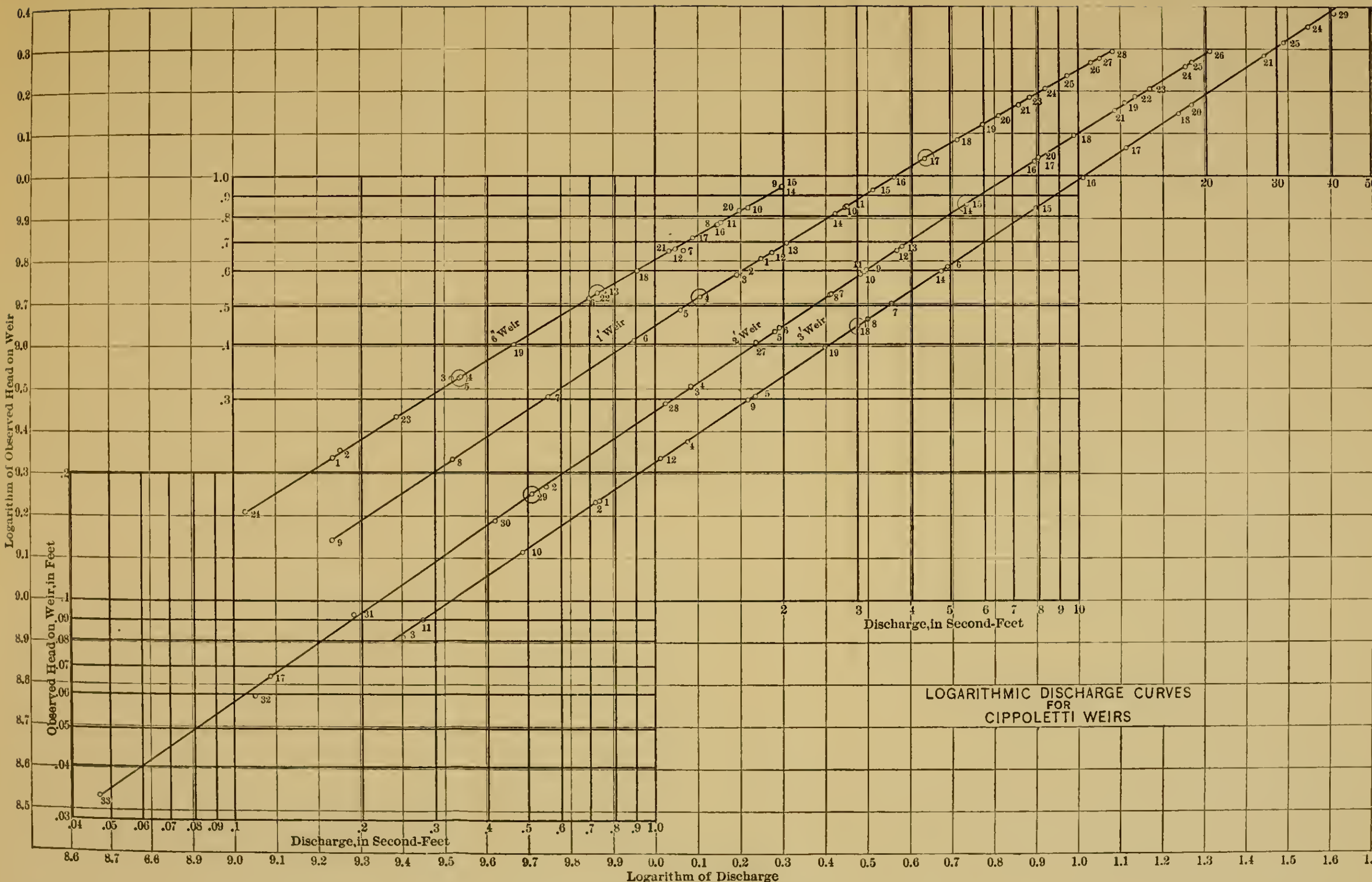


TABLE 4.—GAUGE HEIGHT AND DISCHARGE FOR 2-FOOT CIPPOLETTI WEIR.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00		0.000		0.66		3.635	
0.01		0.01		0.67		3.725	
0.02		0.02		0.68		3.810	
0.03		0.04		0.69		3.895	
0.04		0.05		0.70	3.94	3.98	1.02
0.05	0.08	0.08		0.71		4.06	
0.06		0.10		0.72		4.15	
0.07		0.12		0.73		4.24	
0.08		0.15		0.74		4.33	
0.09		0.18		0.75		4.415	
0.10	0.21	0.21		0.76		4.51	
0.11		0.25		0.77		4.60	
0.12		0.28		0.78		4.69	
0.13		0.32		0.79		4.78	
0.14		0.35		0.80	4.82	4.87	1.04
0.15		0.39		0.81		4.96	
0.16		0.43		0.82		5.05	
0.17		0.47		0.83		5.14	
0.18		0.51		0.84		5.24	
0.19		0.55		0.85		5.34	
0.20	0.60	0.60		0.86		5.14	
0.21		0.65		0.87		5.54	
0.22		0.69		0.88		5.64	
0.23		0.74		0.89		5.74	
0.24		0.79		0.90	5.75	5.84	1.57
0.25		0.84		0.91		5.94	
0.26		0.89		0.92		6.04	
0.27		0.94		0.93		6.14	
0.28		1.00		0.94		6.25	
0.29		1.05		0.95		6.36	
0.30	1.11	1.11		0.96		6.47	
0.31		1.16		0.97		6.68	
0.32		1.22		0.98		6.69	
0.33		1.28		0.99		6.80	
0.34		1.33		1.00	6.73	6.91	2.67
0.35		1.39		1.01		7.02	
0.36		1.45		1.02		7.13	
0.37		1.52		1.03		7.24	
0.38		1.58		1.04		7.35	
0.39		1.64		1.05		7.47	
0.40	1.70	1.70		1.06		7.59	
0.41		1.77		1.07		7.71	
0.42		1.83		1.08		7.83	
0.43		1.90		1.09		7.95	
0.44		1.97		1.10	7.77	8.07	3.86
0.45	2.03	2.03		1.11		8.19	
0.46		2.10		1.12		8.31	
0.47		2.17		1.13		8.43	
0.48		2.24		1.14		8.55	
0.49		2.31		1.15		8.67	
0.50	2.38	2.38		1.16		8.79	
0.51		2.45		1.17		8.91	
0.52		2.53		1.18		9.03	
0.53		2.605		1.19		9.15	
0.54		2.68		1.20	8.85	9.28	1.86
0.55	2.75	2.755	0.18	1.21		9.40	
0.56		2.83		1.22		9.52	
0.57		2.905		1.23		9.65	
0.58		2.98		1.24		9.78	
0.59		3.06		1.25		9.91	
0.60	3.13	3.14	0.32	1.26		10.04	
0.61		3.22		1.27		10.17	
0.62		3.30		1.28		10.30	
0.63		3.38		1.29		10.43	
0.64		3.465		1.30	9.98	10.56	5.81
0.65		3.55		1.31		10.69	

TABLE 4.—(*Continued.*)

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
1.32		10.82		1.67		15.86	
1.33		10.95		1.68		16.02	
1.34		11.08		1.69		16.18	
1.35		11.21		1.70	14.92	16.34	9.52
1.36		11.35		1.71		16.50	
1.37		11.49		1.72		16.66	
1.38		11.63		1.73		16.82	
1.39		11.77		1.74		16.98	
1.40	11.15	11.91	6.82	1.75		17.14	
1.41		12.05		1.76		17.30	
1.42		12.19		1.77		17.46	
1.43		12.33		1.78		17.62	
1.44		12.47		1.79		17.78	
1.45		12.61		1.80	16.26	17.94	10.33
1.46		12.75		1.81		18.10	
1.47		12.89		1.82		18.26	
1.48		13.03		1.83		18.42	
1.49		13.17		1.84		18.58	
1.50	12.37	13.31	7.60	1.85		18.74	
1.51		13.45		1.86		18.91	
1.52		13.59		1.87		19.08	
1.53		13.73		1.88		19.25	
1.54		13.88		1.89		19.42	
1.55		14.03		1.90	17.63	19.59	11.12
1.56		14.18		1.91		19.76	
1.57		14.33		1.92		19.93	
1.58		14.48		1.93		20.10	
1.59		14.63		1.94		20.27	
1.60	13.63	14.78	8.44	1.95		20.45	
1.61		14.93		1.96		20.63	
1.62		15.08		1.97		20.81	
1.63		15.23		1.98		20.99	
1.64		15.38		1.99		21.17	
1.65		15.54		2.00	19.04	21.35	12.13
1.66		15.70					

of the crest the actual discharge is approximately 30% in exeess of the formula discharge. The gauge heights were not carried up to this proportion on the 2-ft. and 3-ft. weirs, but it will be observed that the percentage of difference for the same proportion of gauge height to length of crest agrees fairly well for the different weirs.

Plate XXV shows the logarithmic discharge curves for each Cippoletti weir. For the 6-in., 1-ft., and 2-ft. weirs, it is observed that there are three distinct straight lines for each weir. For each succeeding larger weir, the points plot in less decided breaks, and for the 3-ft. weir, there are only two curves. The equation of each of these separate divisions was taken, and the gauge height at which the change in equation takes place was observed, also the proportional part that each of these gauge heights is of their respective lengths of crest. These are given in Table 7. It will be observed that, for the 6-in. and 1-ft. weirs these breaks occur at gauge heights which are approxi-

TABLE 5.—Gauge Height and Discharge for 3-Foot Cippoletti Weir.

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
0.00		0.00		0.66		5.51	
0.01		0.01		0.67		5.64	
0.02		0.03		0.68		5.77	
0.03		0.05		0.69		5.90	
0.04		0.08		0.70	5.92	6.03	1.86
0.05	0.11	0.12		0.71		6.16	
0.06		0.16		0.72		6.29	
0.07		0.20		0.73		6.42	
0.08		0.24		0.74		6.55	
0.09		0.28		0.75	6.56	6.68	
0.10	0.32	0.33		0.76		6.81	
0.11		0.38		0.77		6.94	
0.12		0.43		0.78		7.08	
0.13		0.48		0.79		7.22	
0.14		0.53		0.80	7.23	7.36	1.80
0.15	0.59	0.59		0.81		7.50	
0.16		0.65		0.82		7.64	
0.17		0.71		0.83		7.78	
0.18		0.77		0.84		7.92	
0.19		0.84		0.85	7.92	8.06	
0.20	0.90	0.91		0.86		8.20	
0.21		0.98		0.87		8.34	
0.22		1.05		0.88		8.48	
0.23		1.12		0.89		8.62	
0.24		1.19		0.90	8.62	8.77	1.74
0.25	1.26	1.27		0.91		8.92	
0.26		1.35		0.92		9.07	
0.27		1.43		0.93		9.22	
0.28		1.51		0.94		9.37	
0.29		1.59		0.95		9.52	
0.30	1.66	1.67		0.96		9.67	
0.31		1.75		0.97		9.82	
0.32		1.84		0.98		9.98	
0.33		1.93		0.99		10.14	
0.34		2.02		1.00	10.10	10.30	1.98
0.35	2.09	2.11		1.01		10.46	
0.36		2.20		1.02		10.62	
0.37		2.29		1.03		10.78	
0.38		2.38		1.04		10.94	
0.39		2.47		1.05		11.10	
0.40	2.56	2.57		1.06		11.26	
0.41		2.67		1.07		11.42	
0.42		2.77		1.08		11.58	
0.43		2.87		1.09		11.74	
0.44		2.97		1.10	11.65	11.90	2.15
0.45	3.05	3.07	0.66	1.11		12.06	
0.46		3.17		1.12		12.22	
0.47		3.27		1.13		12.38	
0.48		3.38		1.14		12.54	
0.49		3.49		1.15		12.71	
0.50	3.57	3.60	0.81	1.16		12.88	
0.51		3.71		1.17		13.05	
0.52		3.82		1.18		13.22	
0.53		3.93		1.19		13.39	
0.54		4.04		1.20	13.28	13.56	2.11
0.55	4.12	4.16		1.21		13.73	
0.56		4.28		1.22		13.91	
0.57		4.40		1.23		14.09	
0.58		4.52		1.24		14.27	
0.59		4.64		1.25		14.45	
0.60	4.69	4.76	1.49	1.26		14.63	
0.61		4.88		1.27		14.81	
0.62		5.00		1.28		14.99	
0.63		5.12		1.29		15.17	
0.64		5.25		1.30	14.97	15.35	2.54
0.65	5.29	5.38		1.31		15.53	

TABLE 5.—(*Continued.*)

Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Cippoletti formula discharge.	Actual discharge.	Percentage of difference.
1.32		15.71		1.89		27.35	
1.33		15.89		1.90	26.45	27.57	4.23
1.34		16.07		1.91		27.79	
1.35	15.84	16.25		1.92		28.01	
1.36		16.44		1.93		28.23	
1.37		16.63		1.94		28.45	
1.38		16.82		1.95		28.67	
1.39		17.01		1.96		28.90	
1.40	16.73	17.20	2.81	1.97		29.13	
1.41		17.39		1.98		29.36	
1.42		17.58		1.99		29.59	
1.43		17.77		2.00	28.57	29.82	4.38
1.44		17.96		2.01		30.05	
1.45		18.15		2.02		30.28	
1.46		18.34		2.03		30.51	
1.47		18.53		2.04		30.74	
1.48		18.72		2.05		30.97	
1.49		18.91		2.06		31.20	
1.50	18.56	19.10	2.91	2.07		31.43	
1.51		19.30		2.08		31.66	
1.52		19.50		2.09		31.89	
1.53		19.70		2.10	30.74	32.13	4.52
1.54		19.90		2.11		32.37	
1.55		20.10		2.12		32.61	
1.56		20.30		2.13		32.85	
1.57		20.50		2.14		33.09	
1.58		20.70		2.15		33.33	
1.59		20.90		2.16		33.57	
1.60	20.44	21.11	3.27	2.17		33.81	
1.61		21.32		2.18		34.05	
1.62		21.53		2.19		34.29	
1.63		21.74		2.20	32.96	34.53	4.76
1.64		21.95		2.21		34.77	
1.65		22.16		2.22		35.01	
1.66		22.37		2.23		35.25	
1.67		22.58		2.24		35.49	
1.68		22.79		2.25		35.73	
1.69		23.00		2.26		35.97	
1.70	22.39	23.21	3.66	2.27		36.21	
1.71		23.42		2.28		36.45	
1.72		23.63		2.29		36.70	
1.73		23.84		2.30	35.23	36.95	4.88
1.74		24.05		2.31		37.20	
1.75		24.27		2.32		37.45	
1.76		24.49		2.33		37.70	
1.77		24.71		2.34		37.95	
1.78		24.93		2.35		38.20	
1.79		25.15		2.36		38.45	
1.80	24.39	25.37	4.02	2.37		38.70	
1.81		25.59		2.38		38.95	
1.82		25.81		2.39		39.20	
1.83		26.03		2.40	37.55	39.45	5.06
1.84		26.25		2.41		39.70	
1.85		26.47		2.42		39.95	
1.86		26.63		2.43		40.20	
1.87		26.91		2.41		40.45	
1.88		27.13		2.45		40.70	

mately one-half the length of the crest and equal to the length of crest, respectively.

For the 2-ft. weir, the heads are not high enough to obtain the break at the length of crest, but the break at about one-half that length is quite evident. This weir makes a change in equation at a lower

proportion than the other two, at approximately one-tenth of the length.

For the 3-ft. weir, there are only two equations, the break occurring at a proportion of gauge height to length of crest of about one-tenth.

Table 1 gives the results of the experiments as recorded, and also, in each run, the value of C in the formula, $Q = Clh^{\frac{3}{2}}$, the customary weir formula.

Table 6 gives the gauge heights of the experiments, arranged in the order of their value, and the corresponding values of C. This table shows that the values of C increase with the gauge height, and that the exponent, $\frac{3}{2}$, or the slope of the logarithmic curve, is not correct, as is also shown by the logarithmic curves of actual discharge.

Table 8 gives the gauge heights for every $\frac{5}{100}$ ft. up to a height of 2.45 ft. These gauge heights are accompanied by the corresponding discharges for each of the four Cippoletti weirs tested. From this table it is seen that the discharges by formula are practically correct up to a gauge height of one-third of the length of the crest. Above this point, however, the use of a new formula or discharge table is essential to accuracy. This table also shows that, above the limits of the formula, the weirs do not discharge in proportion to their length of crest, and that this difference increases with the gauge height. It will be observed that the discharges of the 2-ft. and 3-ft. weirs are more in proportion to the length of the crest than any of the others.

Table 9 shows the values of C in the weir formula, $Q = Clh^{\frac{3}{2}}$ for all the Cippoletti weirs. The values were prepared for each $\frac{5}{100}$ ft. of gauge height, up to a head of 0.3 ft., and for every $\frac{1}{10}$ ft. above that point. The values of Q in the formula were taken from the prepared discharge tables. This table shows that above a gauge height equal to one-third of the length of the crest, C increases very rapidly in each case, and that this increase is not uniform for all the weirs, but is more nearly in proportion to the ratio of gauge height to length of crest.

The foregoing tables and plates show that the Cippoletti weir formula may be used with reasonable accuracy for depths up to one-third of the length of the crest. Above this gauge height, it will be necessary to use tables prepared carefully from actual ratings. The

TABLE 6.—Gauge Height and Values of C in the Formula, $Q = Clh^{\frac{3}{2}}$

Cippoletti Weirs.

Series No.	Run No.	Average gauge height.	C.
6-Inch Weir.			
1	24	0.161	3.287
	1	0.217	3.362
	2	0.225	3.164
	23	0.270	3.478
	3	0.334	3.304
	5	0.334	3.552
	4	0.336	3.562
	19	0.400	3.676
	6	0.513	3.783
	22	0.530	3.795
	13	0.537	3.867
	18	0.600	3.898
	7	0.667	4.284
	21	0.667	3.954
	12	0.675	4.028
	17	0.715	4.055
	8	0.770	4.132
	16	0.774	4.126
	11	0.776	4.166
	20	0.830	4.181
	10	0.839	4.291
	14	0.940	4.338
	9	0.941	4.329
	15	0.945	4.337
1-Foot Weir.			
7	9	0.138	3.333
	8	0.215	3.320
	7	0.302	3.361
	6	0.409	3.394
	5	0.482	3.422
	4	0.518	3.404
	3	0.588	3.460
	2	0.594	3.454
	1	0.637	3.487
	12	0.659	3.520
	13	0.693	3.530
	14	0.815	3.595
	10	0.840	3.599
	11	0.846	3.604
	15	0.927	3.656
	16	0.992	3.692
	17	1.097	3.758
	18	1.219	3.835
	19	1.320	3.913
	20	1.386	3.971
	21	1.476	4.034
	22	.....	.....
	23	1.528	4.076
	24	1.604	4.121
	25	1.721	4.186
	26	1.853	4.265
	27	1.897	4.312
	28	1.978	4.366
2-Foot Weir.			
8	33	0.034	3.730
	32	0.059	3.889
2-Foot Weir.—(*Continued.*)			
8	1	0.066	3.588
	31	0.092	3.459
	30	0.154	3.455
	29	0.178	3.409
	2	0.186	3.441
	28	0.290	3.374
	3	0.317	3.392
	4	0.318	3.380
	27	0.406	3.338
	5	0.430	3.386
	6	0.438	3.368
	8	0.524	3.385
	7	0.526	3.385
	11	0.588	3.371
	10	0.594	3.384
	9	0.601	3.380
	12	0.668	3.411
	13	0.679	3.197
	14	0.853	3.445
	15	0.854	3.448
	16	1.086	3.499
	17	1.090	3.514
	20	1.101	3.494
	18	1.238	3.557
	21	1.425	3.601
	19	1.484	3.628
	22	1.537	3.607
	23	1.598	3.689
	24	1.816	3.719
	25	1.852	3.722
	26	1.987	3.737
3-Foot Weir.			
11	3	0.082	3.514
	11	0.090	3.469
	10	0.130	3.461
	2	0.170	3.433
	1	0.172	3.436
	12	0.216	3.410
	4	0.236	3.450
	9	0.297	3.393
	5	0.302	3.430
	19	0.396	3.357
	13	0.444	3.366
	8	0.460	3.386
	7	0.500	3.400
	14	0.598	3.400
	6	0.609	3.414
	15	0.844	3.413
	16	0.994	3.445
	17	1.167	3.435
	18	1.410	3.463
	20	1.470	3.489
	21	1.923	3.502
	22	.....	.....
	25	2.065	3.526
	24	2.216	3.532
	23	2.415	3.676

TABLE 7.—EQUATIONS AND LOCATION OF BREAKS IN LOGARITHMIC CURVES.

CIPPOLETTI WEIRS.

Equation.	Gauge height of break.	Gauge height of break divided by length of weir crest.
6-INCH WEIR.		
$4.000\ lh^{1.597}$	.....	.....
$4.178\ lh^{1.633}$	0.327	0.654
$4.409\ lh^{1.71}$	0.530	1.060
1-FOOT WEIR.		
$3.467\ lh^{1.515}$	.....	.....
$3.690\ lh^{1.61}$	0.518	0.518
$3.631\ lh^{1.75}$	1.109	1.109
2-FOOT WEIR.		
$3.083\ lh^{1.44}$	.....	.....
$3.388\ lh^{1.50}$	0.178	0.089
$3.443\ lh^{1.63}$	0.864	0.432
3-FOOT WEIR.		
$3.333\ lh^{1.49}$	.....	.....
$3.435\ lh^{1.526}$	0.444	0.148

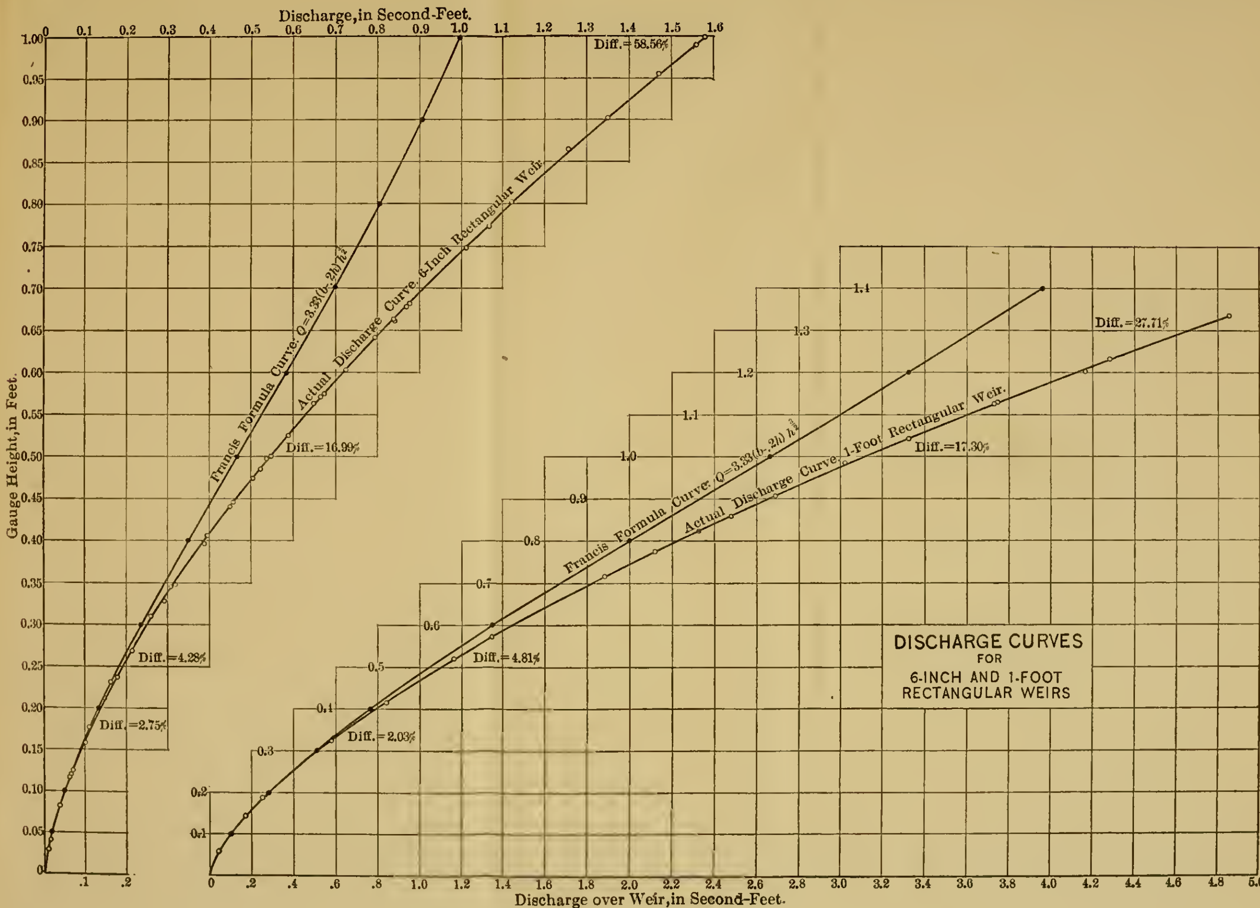
results show that the discharge curves of smaller weirs above a gauge height of one-third of the length of the crest are not a single section of a parabola, but a series of short sections of parabolas. This series of parabolas, however, seems to become less evident as the weirs increase in length, and it will be assumed that the logarithmic curve for the 3-ft. weir above a gauge height of 0.444 ft. is a good representative of the behavior of the larger weirs, and that it may be used with reasonable accuracy up to a depth equal to the length of the weir crest. Thus, in the use of Cippoletti weirs, the formula will be taken as correct up to gauge heights of one-third of the length of the crest. Above this point, for 3-ft. weirs and smaller, the tables given here-

with are recommended, and for weirs of greater length than 3 ft., for gauge heights of more than one-third of the length of the crest, the formula, 3.435 $lh^{1.526}$, should be used. In this manner accurate results are assured for all reasonable gauge heights.

Rectangular Weirs.—The experiments on sharp-crested rectangular weirs were made on the same sizes as were those of the Cippoletti type, namely, 6-in., 1-ft., 2-ft., and 3-ft. These were tested and calibrated at the same headings and in the same manner as the

TABLE 8.—GAUGE HEIGHTS AND DISCHARGE FOR CIPPOLETTI WEIRS.

Gauge height, in feet.	Discharge of 6-in. weir.	Discharge of 1-ft. weir.	Discharge of 2-ft. weir.	Discharge of 3-ft. weir.
0.00	0.000	0.000	0.000	0.00
0.05	0.019	0.038	0.08	0.12
0.10	0.053	0.107	0.21	0.33
0.15	0.098	0.196	0.39	0.59
0.20	0.151	0.301	0.60	0.91
0.25	0.214	0.421	0.84	1.27
0.30	0.288	0.553	1.11	1.67
0.35	0.371	0.697	1.39	2.11
0.40	0.463	0.852	1.70	2.57
0.45	0.563	1.016	2.03	3.07
0.50	0.669	1.200	2.38	3.60
0.55	0.781	1.396	2.755	4.16
0.60	0.906	1.603	3.14	4.76
0.65	1.043	1.824	3.55	5.38
0.70	1.186	2.066	3.98	6.03
0.75	1.336	2.312	4.415	6.68
0.80	1.492	2.565	4.87	7.36
0.85	1.657	2.830	5.34	8.06
0.90	1.832	3.105	5.84	8.77
0.95	2.012	3.389	6.36	9.52
1.00	2.194	3.686	6.91	10.30
1.05	......	3.999	7.47	11.10
1.10	......	4.325	8.07	11.90
1.15	......	4.670	8.67	12.71
1.20	......	5.027	9.28	13.56
1.25	......	5.395	9.91	14.45
1.30	......	5.781	10.56	15.35
1.35	......	6.183	11.21	16.25
1.40	......	6.595	11.91	17.20
1.45	......	7.015	12.61	18.15
1.50	......	7.450	13.31	19.10
1.55	......	7.890	14.03	20.10
1.60	......	8.343	14.78	21.11
1.65	......	8.806	15.54	22.16
1.70	......	9.276	16.34	23.21
1.75	......	9.746	17.14	24.27
1.80	......	10.229	17.94	25.37
1.85	......	10.728	18.74	26.47
1.90	......	11.256	19.59	27.57
1.95	......	11.791	20.45	28.67
2.00	.....	12.345	21.35	29.82
2.05	......	......	.....	30.97
2.10	......	......	......	32.13
2.15	......	......	......	33.33
2.20	......	......	.....	34.53
2.25	......	......	......	35.73
2.30	......	......	......	36.95
2.35	......	......	......	38.20
2.40	......	......	......	39.45
2.45	......	......	......	40.70



DISCHARGE CURVES
FOR
6-INCH AND 1-FOOT
RECTANGULAR WEIRS

trapezoidal weirs. No velocity of approach existed in any case. Full contraction and a perfect, undisturbed, and free nappe were maintained in every run. The different compartments of the measuring tank were used as the heads and discharge varied.

TABLE 9.—GAUGE HEIGHT AND VALUES OF C IN THE FORMULA, $Q = Clh^{\frac{3}{2}}$ FOR THE CIPPOLETTI WEIR.

Gauge height.	Length of weir, in feet.			
	6-in.	1-ft.	2-ft.	3-ft.
0.0				
0.05	3.367	3.367	3.367	3.571
0.10	3.367	3.367	3.367	3.481
0.15	3.367	3.567	3.367	3.367
0.20	3.367	3.367	3.367	3.393
0.25	3.424	3.367	3.367	3.387
0.30	3.506	3.367	3.367	3.388
0.40	3.660	3.367	3.367	3.386
0.50	3.781	3.394	3.367	3.394
0.60	3.898	3.419	3.377	3.413
0.70	4.049	3.527	3.398	3.431
0.80	4.170	3.585	3.403	3.428
0.90	4.291	3.637	3.429	3.424
1.00	4.388	3.686	3.455	3.433
1.10	......	3.749	3.497	3.438
1.20	......	3.824	3.530	3.438
1.30	......	3.900	3.562	3.452
1.40	......	3.981	3.595	3.461
1.50	......	4.053	3.621	3.464
1.60	......	4.122	3.651	3.477
1.70	......	4.186	3.687	3.491
1.80	......	4.235	3.714	3.501
1.90	......	4.298	3.740	3.509
2.00	......	4.365	3.775	3.515
2.10	......	......	......	3.519
2.20	......	......	......	3.527
2.30	......	......	......	3.531
2.40	......	......	......	3.537

Tables and plates similar to those for the Cippoletti weirs were prepared for this type. The Francis formula for rectangular weirs was used, and in each case the percentage of difference between the actual and Francis discharges was tabulated.

Table 10 gives the actual experimental data for each weir, together with the corresponding values of C in the formula, $Q = Clh^{\frac{3}{2}}$.

Tables 11 to 14 give the actual discharges corresponding to each $\frac{1}{100}$ ft. of gauge height, up to the limit of the experiments. The discharges, computed from the Francis formula, $Q = 3.333\ (l - 0.2h)\ h^{\frac{3}{2}}$, are tabulated for each $\frac{1}{10}$ ft. of gauge height, and the percentage of difference between this and the actual discharge is recorded as shown. It is seen that the formula is only fairly correct for very low heads.

TABLE 10.—Results of Experiments on Weirs.
Rectangular Weirs.

Series No.	Run No.	Average gauge height, in feet.	Variation in gauge height.	Length of weir, in inches.	Time interval, in seconds.	Actual discharge, in cubic feet.	Discharge in second-feet.	C in formula $Q = Clh^{\frac{3}{2}}$
		h		l			Q	
1	1	0.117	0.018	6	1 200	77	0.064	3.192
	2	0.112	0.000	6	1 200	75	0.062	3.307
	3	0.156	0.012	6	1 200	120	0.100	3.247
	4	0.236	0.006	6	900	162	0.180	3.139
	5	0.348	0.004	6	600	194	0.323	3.147
	6	0.499	0.006	6	600	331	0.552	3.132
	7	0.170	0.006	6	1 200	129	0.108	3.081
	8	0.082	0.006	6	2 400	93	0.039	3.319
	9	0.042	0.000	6	3 600	52	0.014	3.256
	10	0.030	0.006	6	3 600	31	0.009	3.462
	11	0.678	0.019	6	600	524	0.873	3.128
	12	0.574	0.006	6	600	406	0.677	3.113
	13	0.774	0.005	6	600	642	1.070	3.142
	14	0.866	0.015	6	600	757	1.262	3.132
	15	0.992	0.019	6	540	845	1.565	3.168
	16	0.751	0.000	6	600	612	1.020	3.135
	17	0.803	0.006	6	600	678	1.130	3.141
	18	0.902	0.001	6	600	813	1.355	3.163
	19	0.955	0.004	6	600	884	1.473	3.157
	20	0.999	0.018	6	600	949	1.582	3.169
	21	0.682	0.030	6	600	528	0.880	3.125
	22	0.572	0.002	6	420	282	0.671	3.102
	23	0.564	0.007	6	600	392	0.653	3.083
	24	0.487	0.006	6	600	317	0.528	3.108
	25	0.446	0.009	6	600	276	0.460	3.088
	26	0.405	0.006	6	600	239	0.398	3.088
	27	0.346	0.003	6	600	187	0.312	3.065
	28	0.309	0.004	6	600	157	0.262	3.050
	29	0.231	0.019	6	600	100	0.167	3.009
	30	0.110	0.010	6	2 400	143	0.060	3.288
	31	0.154	0.006	6	1 800	173	0.096	3.174
	32	0.208	0.004	6	900	134	0.149	3.143
	33	0.604	0.005	6	600	439	0.732	3.119
	34	0.524	0.007	6	600	356	0.593	3.127
	35	0.661	0.013	6	600	507	0.845	3.145
	36	0.472	0.006	6	600	303	0.505	3.114
	37	0.662	0.012	6	600	506	0.843	3.130
	38	0.638	0.012	6	600	479	0.798	3.132
	39	0.498	0.007	6	600	326	0.543	3.090
	40	0.442	0.003	6	600	274	0.457	3.110
	41	0.398	0.010	6	600	234	0.390	3.106
	42	0.328	0.011	6	600	176	0.293	3.119
	43	0.268	0.006	6	900	194	0.216	3.112
13	1	1.338	0.009	12	600	2 922	4.870	3.147
	2	1.126	0.018	12	300	1 123	3.743	3.133
	3	1.205	0.004	12	600	2 506	4.177	3.158
	4	1.130	0.009	12	600	2 254	3.757	3.128
	5	1.044	0.010	12	600	1 999	3.332	3.124
	6	0.982	0.013	12	600	1 815	3.025	3.108
	7	0.909	0.002	12	600	1 617	2.695	3.109
	8	0.857	0.005	12	600	1 490	2.483	3.130
	9	0.775	0.008	12	600	1 278	2.130	3.122
	10	0.571	0.012	12	600	814	1.357	3.146
	11	0.714	0.005	12	600	1 131	1.885	3.124
	12	0.518	0.009	12	600	702	1.170	3.138
	13	0.416	0.003	12	900	762	0.847	3.157
	14	0.189	0.010	12	1 200	320	0.267	3.248
	15	0.324	0.005	12	1 200	703	0.586	3.178
	16	0.056	0.009	12	1 500	67	0.045	3.383
	17	0.142	0.005	12	1 800	321	0.178	3.327
	18	0.102	0.015	12	1 020	104	0.102	3.129
	19	0.822	0.011	12	600	1 400	2.333	3.131
	20	1.233	0.008	12	480	2 060	4.292	3.135

TABLE 10.—(*Continued.*)

Series No.	Run No.	Average gauge height, in feet.	Variation in gauge height.	Length of weir.	Time interval, in seconds.	Actual discharge, cubic feet.	Discharge, in second-feet.	C in formula $Q = Clh^{\frac{3}{2}}$
		h		l			Q	
14	1	0.081	0.002	24	1 800	290	0.161	3.500
	2	0.104	0.008	24	1 200	277	0.231	3.438
	3	0.167	0.003	24	900	416	0.462	3.382
	4	0.233	0.005	24	600	419	0.748	3.324
	5	0.294	0.002	24	600	627	1.045	3.278
	6	0.395	0.008	24	600	970	1.617	3.256
	7	0.510	0.020	24	600	1 417	2.362	3.243
	8	0.573	0.003	24	600	1 669	2.782	3.207
	9	0.717	0.011	24	600	2 321	3.868	3.186
	10	0.910	0.003	24	480	2 635	5.490	3.162
	11	1.102	0.004	24	420	3 063	7.293	3.152
	12	......	......	..	......	......	......	......
	13	1.283	0.011	24	300	2 724	9.080	3.124
	14	1.429	0.015	24	300	3 171	10.570	3.091
	15	0.549	0.002	24	900	2 326	2.584	3.176
	16	0.886	0.007	24	900	4 716	5.240	3.141
	17	1.185	0.005	24	600	4 857	8.095	3.138
	18	1.574	0.005	24	480	5 934	12.362	3.130
	19	0.059	0.009	24	2 100	202	0.096	3.331
	20	0.214	0.004	24	600	394	0.657	3.318
15	1	0.482	0.000	36	600	2 001	3.335	3.322
	2	0.468	0.002	36	600	1 921	3.202	3.333
	3	0.426	0.001	36	630	1 754	2.784	3.337
	4	0.385	0.000	36	600	1 448	2.413	3.367
	5	......	......	36	......	......	......	......
	6	0.101	0.000	36	930	316	0.340	3.531
	7	0.310	0.001	36	600	1 028	1.713	3.308
	8	0.198	0.001	36	900	808	0.898	3.398
	9	0.452	0.001	36	600	1 800	3.000	3.291
	10	0.611	0.002	36	390	1 836	4.708	3.262
	11	0.950	0.003	36	480	4 325	9.010	3.244
	12	0.826	0.003	36	600	4 401	7.335	3.257
	13	1.190	0.007	36	510	6 398	12.545	3.221
	14	2.058	0.004	36	360	10 077	27.992	3.160
	15	2.061	0.005	36	300	8 409	28.030	3.157
	16	1.820	0.007	36	360	8 340	23.167	3.145
	17	1.606	0.010	36	300	5 802	19.340	3.167
	18	1.401	0.003	36	360	5 705	15.847	3.185
	19	1.382	0.004	36	600	9 353	15.588	3.198
	20	1.609	0.004	36	360	6 985	19.403	3.169
	21	0.690	0.001	36	600	3 358	5.597	3.255

Table 15 gives the value of C in the formula, $Q = Clh^{\frac{3}{2}}$, for the gauge heights of the various experiments, arranged in the order of their value. It will be observed that these do not show the marked variation characterizing the Cippoletti weirs. Table 16 gives the equations of the different parabolas, the gauge height at which the equations change, and the proportion that these gauge heights are of the lengths of crest. It may be seen that the equations do not have such a wide range as in the previous case, and that the three equations agree more closely with each other as the size of the weir increases, and finally

TABLE 11.—Gauge Height and Discharge for 6-Inch Rectangular Weir.

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.000	0.000		0.51		0.568	
0.01		0.002		0.52		0.585	
0.02		0.005		0.53		0.602	
0.03		0.009		0.54		0.619	
0.04		0.013		0.55		0.636	
0.05	0.018	0.018		0.56		0.653	
0.06		0.024		0.57		0.671	
0.07		0.030		0.58		0.689	
0.08		0.037		0.59		0.707	
0.09		0.044		0.60	0.588	0.725	23.30
0.10	0.050	0.051	2.00	0.61		0.743	
0.11		0.059		0.62		0.761	
0.12		0.067		0.63		0.780	
0.13		0.075		0.64		0.799	
0.14		0.084		0.65		0.818	
0.15		0.093		0.66		0.837	
0.16		0.102		0.67		0.856	
0.17	0.109	0.112	2.75	0.68		0.876	
0.18		0.122		0.69		0.896	
0.19		0.132		0.70	0.702	0.916	30.49
0.20	0.137	0.142	3.65	0.71		0.936	
0.21		0.152		0.72		0.956	
0.22		0.162		0.73		0.976	
0.23		0.173		0.74		0.997	
0.24		0.184		0.75		1.018	
0.25		0.195		0.76		1.039	
0.26		0.206		0.77		1.060	
0.27		0.218		0.78		1.081	
0.28		0.230		0.79		1.103	
0.29		0.242		0.80	0.810	1.125	38.89
0.30	0.241	0.254	5.39	0.81		1.147	
0.31		0.266		0.82		1.169	
0.32		0.279		0.83		1.191	
0.33		0.292		0.84		1.213	
0.34		0.305		0.85		1.235	
0.35		0.318		0.86		1.258	
0.36		0.332		0.87		1.281	
0.37		0.346		0.88		1.304	
0.38		0.361		0.89		1.327	
0.39		0.376		0.90	0.910	1.350	48.35
0.40	0.354	0.391	10.45	0.91		1.373	
0.41		0.406		0.92		1.396	
0.42		0.421		0.93		1.419	
0.43		0.437		0.94		1.442	
0.44		0.453		0.95		1.465	
0.45	0.412	0.469		0.96		1.488	
0.46		0.485		0.97		1.512	
0.47		0.501		0.98		1.536	
0.48		0.517		0.99		1.560	
0.49		0.534		1.00	0.999	1.584	58.56
0.50	0.471	0.551	16.99				

become one parabola in the 3-ft. weir. The breaks occur at about the same relative positions as in the Cippoletti weirs.

Table 17 gives the discharges of each weir for every $\frac{5}{100}$ ft. of gauge height, as far as the experiments were carried. This table shows that these results bear a very marked proportionality to the various lengths of crest.

PLATE XXVII.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1913.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.

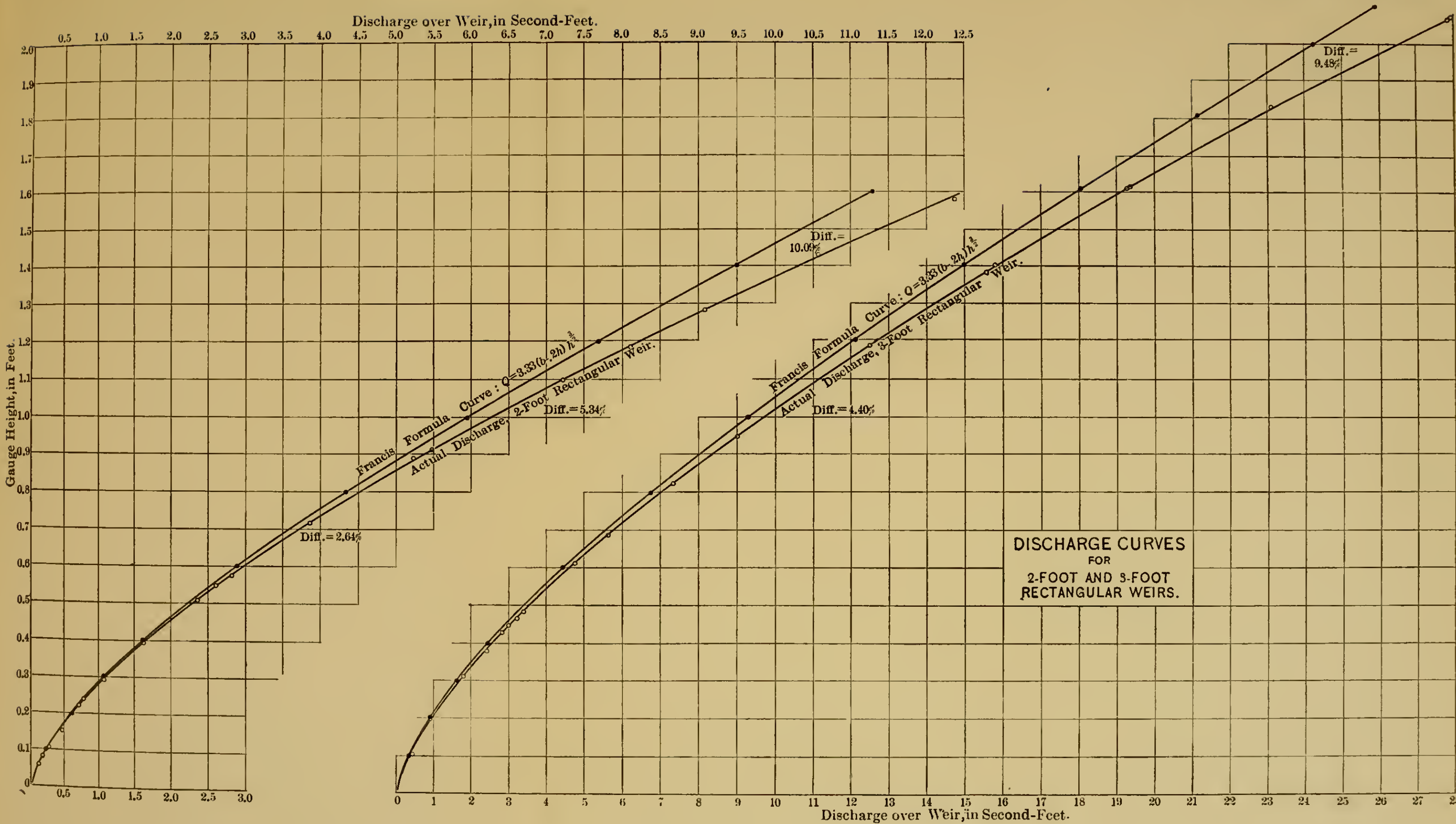


TABLE 12.—GAUGE HEIGHT AND DISCHARGE FOR 1-FOOT RECTANGULAR WEIR.

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.000	0.000		0.65		1.640	
0.01		0.005		0.66		1.677	
0.02		0.010		0.67		1.715	
0.03		0.017		0.68		1.753	
0.04		0.026		0.69		1.791	
0.05	0.037	0.037		0.70	1.677	1.830	9.12
0.06		0.048		0.71		1.869	
0.07		0.060		0.72		1.908	
0.08		0.073		0.73		1.948	
0.09		0.087		0.74		1.988	
0.10	0.103	0.103		0.75		2.028	
0.11		0.119		0.76		2.068	
0.12		0.136		0.77		2.108	
0.13		0.153		0.78		2.149	
0.14		0.170		0.79		2.191	
0.15	0.188	0.188		0.80	2.001	2.233	11.59
0.16		0.206		0.81		2.276	
0.17		0.225		0.82		2.319	
0.18		0.245		0.83		2.362	
0.19		0.266		0.84		2.405	
0.20	0.286	0.287	0.35	0.85		2.448	
0.21		0.308		0.86		2.492	
0.22		0.330		0.87		2.536	
0.23		0.352		0.88		2.580	
0.24		0.375		0.89		2.624	
0.25		0.398		0.90	2.331	2.668	14.46
0.26		0.421		0.91		2.712	
0.27		0.445		0.92		2.757	
0.28		0.470		0.93		2.802	
0.29		0.495		0.94		2.847	
0.30	0.511	0.521	1.36	0.95		2.892	
0.31		0.547		0.96		2.938	
0.32		0.574		0.97		2.984	
0.33	0.590	0.602	2.03	0.98		3.031	
0.34		0.630		0.99		3.078	
0.35		0.658		1.00	2.664	3.125	17.30
0.36		0.686		1.01		3.172	
0.37		0.714		1.02		3.219	
0.38		0.743		1.03		3.266	
0.39		0.772		1.04		3.314	
0.40	0.775	0.801	3.35	1.05		3.362	
0.41		0.830		1.06		3.411	
0.42		0.860		1.07		3.460	
0.43		0.890		1.08		3.509	
0.44		0.920		1.09		3.559	
0.45		0.950		1.10	2.997	3.609	20.42
0.46		0.981		1.11		3.659	
0.47		1.012		1.12		3.709	
0.48		1.044		1.13		3.759	
0.49		1.077		1.14		3.809	
0.50	1.060	1.111	4.81	1.15		3.860	
0.51		1.145		1.16		3.911	
0.52		1.179		1.17		3.962	
0.53		1.213		1.18		4.014	
0.54		1.247		1.19		4.066	
0.55		1.282		1.20	3.327	4.119	23.81
0.56		1.317		1.21		4.172	
0.57		1.352		1.22		4.225	
0.58		1.387		1.23		4.278	
0.59		1.423		1.24		4.332	
0.60	1.362	1.459	7.12	1.25		4.387	
0.61		1.495		1.26		4.442	
0.62		1.531		1.27		4.497	
0.63		1.557		1.28		4.552	
0.64		1.603		1.29		4.608	

TABLE 12.—(*Continued.*)

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
1.30	3.652	4.664	27.71	1.36		....	
1.31		4.720		1.37		....	
1.32		4.777		1.38		....	
1.33		4.834		1.39		....	
1.34		....		1.40	3.972	....	
1.35	3.813	....					

TABLE 13.—GAUGE HEIGHT AND DISCHARGE FOR 2-FOOT RECTANGULAR WEIR.

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.00	0.00		0.51		2.34	
0.01		0.01		0.52		2.41	
0.02		0.025		0.53		2.48	
0.03		0.04		0.54		2.55	
0.04		0.06		0.55		2.62	
0.05	0.08	0.08		0.56		2.69	
0.06		0.10		0.57		2.76	
0.07		0.125		0.58		2.83	
0.08		0.15		0.59		2.90	
0.09		0.18		0.60	2.91	2.98	2.41
0.10	0.21	0.21		0.61		3.05	
0.11		0.24		0.62		3.12	
0.12		0.27		0.63		3.19	
0.13		0.30		0.64		3.26	
0.14		0.34		0.65		3.34	
0.15		0.38		0.66		3.42	
0.16		0.42		0.67	3.41	3.50	2.64
0.17		0.46		0.68		3.58	
0.18		0.50		0.69		3.66	
0.19		0.54		0.70	3.63	3.74	3.03
0.20	0.58	0.58		0.71		3.82	
0.21		0.63		0.72		3.90	
0.22		0.68		0.73		3.98	
0.23		0.73		0.74		4.06	
0.24		0.78		0.75		4.14	
0.25		0.83		0.76		4.22	
0.26		0.88		0.77		4.30	
0.27		0.93		0.78		4.38	
0.28		0.98		0.79		4.46	
0.29		1.03		0.80	4.38	4.54	3.65
0.30	1.06	1.08	1.89	0.81		4.62	
0.31		1.13		0.82		4.70	
0.32		1.18		0.83		4.78	
0.33		1.23		0.84		4.87	
0.34		1.29		0.85		4.96	
0.35		1.35		0.86		5.05	
0.36		1.41		0.87		5.14	
0.37		1.47		0.88		5.23	
0.38		1.53		0.89		5.32	
0.39		1.59		0.90	5.17	5.41	4.64
0.40	1.62	1.65	1.85	0.91		5.50	
0.41		1.71		0.92		5.59	
0.42		1.77		0.93		5.68	
0.43		1.83		0.94		5.77	
0.44		1.89		0.95		5.86	
0.45		1.95		0.96		5.95	
0.46		2.01		0.97		6.04	
0.47		2.07		0.98		6.13	
0.48		2.13		0.99		6.22	
0.49		2.20		1.00	5.99	6.31	5.34
0.50	2.24	2.27	1.34	1.01		6.40	

PLATE XXVIII.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1913.
STEWARD AND LONGWELL ON
WEIR DISCHARGE EXPERIMENTS.

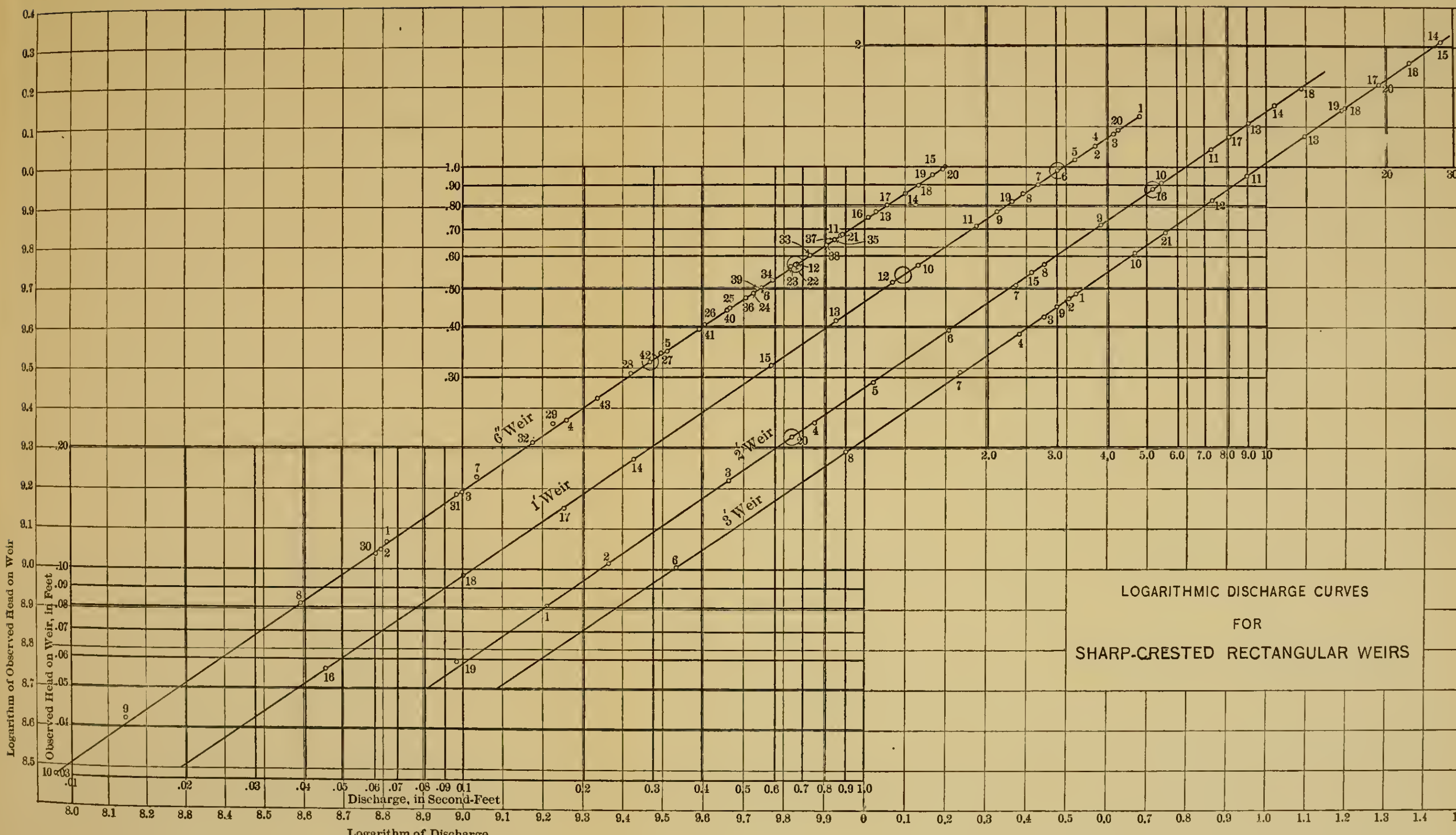


TABLE 13.—(*Continued.*)

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
1.02		6.49		1.30	8.59	9.27	7.92
1.03		6.58		1.31		9.37	
1.04		6.67		1.32		9.47	
1.05		6.76		1.33		9.58	
1.06		6.85		1.34		9.69	
1.07		6.95		1.35		9.80	
1.08		7.05		1.36		9.91	
1.09		7.15		1.37		10.02	
1.10	6.84	7.25	5.99	1.38		10.13	
1.11		7.35		1.39		10.24	
1.12		7.45		1.40	9.49	10.35	9.06
1.13		7.55		1.41		10.46	
1.14		7.65		1.42		10.57	
1.15		7.75		1.43		10.68	
1.16		7.85		1.44		10.79	
1.17		7.95		1.45		10.90	
1.18		8.05		1.46		11.01	
1.19		8.15		1.47		11.12	
1.20	7.70	8.25	7.14	1.48		11.23	
1.21		8.35		1.49		11.34	
1.22		8.45		1.50	10.40	11.45	10.09
1.23		8.55		1.51		11.56	
1.24		8.65		1.52		11.67	
1.25		8.75		1.53		11.78	
1.26		8.85		1.54		11.89	
1.27		8.95		1.55		12.00	
1.28		9.05		1.56			
1.29		9.15					

Table 18 gives the values of C in the formula, $Q = Clh^{\frac{3}{2}}$, for each of the rectangular weirs, for each $\frac{5}{100}$ ft. of gauge height to 0.3 ft. head, and for each $\frac{1}{10}$ ft. thereafter. These values are based on the discharge, Q, as given in Tables 11 to 14. It will be seen that these values are also variable, both as to gauge height and length of weir crest, thus again indicating the fallacy of the original formula, $Q = Clh^{\frac{3}{2}}$. Table 19 contains the values of C of Table 18 for weirs up to 3 ft. in length and values as given by the late Hamilton Smith, M. Am. Soc. C. E., for the larger weirs. The tables and plates given herewith indicate that the Francis formula is not applicable, within reasonable limits of accuracy, for gauge heights of more than one-fourth of the length of the crest, and also that the errors increase with the size of the weir. The varying values of C indicate, as in the case of the Cippoletti weirs, that the exponent, $\frac{3}{2}$, in the formula, $Q = Clh^{\frac{3}{2}}$, is incorrect, and that if this formula is used it will be necessary to obtain a new value of C for each computation of discharge. Owing to the proportionality existing among the various values of discharge for the same gauge heights, and the tendency toward one equation

TABLE 14.—GAUGE HEIGHT AND DISCHARGE FOR 3-FOOT RECTANGULAR WEIR.

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
0.00	0.00	0.00		0.65		5.13	
0.01		0.01		0.66		5.25	
0.02		0.04		0.67		5.37	
0.03		0.07		0.68		5.49	
0.04		0.10		0.69		5.61	
0.05		0.13		0.70	5.58	5.73	2.69
0.06		0.17		0.71		5.85	
0.07		0.21		0.72		5.97	
0.08		0.25		0.73		6.09	
0.09		0.29		0.74		6.22	
0.10	0.31	0.33	6.45	0.75		6.35	
0.11		0.38		0.76		6.48	
0.12		0.43		0.77		6.61	
0.13		0.48		0.78		6.74	
0.14		0.54		0.79		6.87	
0.15		0.60		0.80	6.77	7.00	3.40
0.16		0.66		0.81		7.13	
0.17		0.72		0.82		7.26	
0.18		0.78		0.83		7.39	
0.19		0.84		0.84		7.52	
0.20	0.88	0.91	3.41	0.85		7.65	
0.21		0.98		0.86		7.78	
0.22		1.05		0.87		7.91	
0.23		1.12		0.88		8.05	
0.24		1.19		0.89		8.19	
0.25		1.26		0.90	8.02	8.33	3.87
0.26		1.33		0.91		8.47	
0.27		1.41		0.92		8.61	
0.28		1.49		0.93		8.75	
0.29		1.57		0.94		8.89	
0.30	1.61	1.65	2.48	0.95		9.03	
0.31		1.73		0.96		9.17	
0.32		1.81		0.97		9.31	
0.33		1.90		0.98		9.45	
0.34		1.99		0.99		9.59	
0.35		2.08		1.00	9.32	9.73	4.40
0.36		2.17		1.01		9.87	
0.37		2.26		1.02		10.01	
0.38		2.35		1.03		10.15	
0.39		2.44		1.04		10.30	
0.40	2.46	2.53	2.85	1.05		10.45	
0.41		2.62		1.06		10.60	
0.42		2.71		1.07		10.75	
0.43		2.80		1.08		10.90	
0.44		2.90		1.09		11.05	
0.45		3.00		1.10	10.68	11.20	4.87
0.46		3.10		1.11		11.35	
0.47		3.20		1.12		11.50	
0.48		3.30		1.13		11.65	
0.49		3.40		1.14		11.80	
0.50	3.41	3.50	2.64	1.15		11.95	
0.51		3.60		1.16		12.10	
0.52		3.70		1.17		12.25	
0.53		3.80		1.18		12.40	
0.54		3.90		1.19		12.55	
0.55		4.01		1.20	12.08	12.70	5.13
0.56		4.12		1.21		12.85	
0.57		4.23		1.22		13.00	
0.58		4.34		1.23		13.15	
0.59		4.45		1.24		13.31	
0.60	4.46	4.56	2.24	1.25		13.47	
0.61		4.67		1.26		13.63	
0.62		4.78		1.27		13.79	
0.63		4.89		1.28		13.95	
0.64		5.01		1.29		14.11	

TABLE 14.—(*Continued.*)

Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.	Gauge height.	Francis formula discharge.	Actual discharge.	Percentage of difference.
1.30	13.52	14.27	5.55	1.71		21.19	
1.31		14.43		1.72		21.37	
1.32		14.59		1.73		21.55	
1.33		14.75		1.74		21.73	
1.34		14.91		1.75		21.91	
1.35		15.07		1.76		22.10	
1.36		15.23		1.77		22.29	
1.37		15.39		1.78		22.48	
1.38		15.55		1.79		22.67	
1.39		15.71		1.80	21.23	22.86	7.68
1.40	15.00	15.87	5.8	1.81		23.05	
1.41		16.03		1.82		23.24	
1.42		16.19		1.83		23.43	
1.43		16.35		1.84		23.62	
1.44		16.51		1.85		23.81	
1.45		16.68		1.86		24.00	
1.46		16.85		1.87		24.20	
1.47		17.02		1.88		24.40	
1.48		17.19		1.89		24.60	
1.49		17.36		1.90	22.85	24.80	8.53
1.50	16.53	17.53	6.05	1.91		25.00	
1.51		17.70		1.92		25.20	
1.52		17.87		1.93		25.40	
1.53		18.04		1.94		25.60	
1.54		18.21		1.95		25.80	
1.55		18.38		1.96		26.00	
1.56		18.55		1.97		26.20	
1.57		18.72		1.98		26.40	
1.58		18.89		1.99		26.60	
1.59		19.06		2.00	24.48	26.80	9.48
1.60	18.06	19.23	6.48	2.01		27.00	
1.61		19.40		2.02		27.20	
1.62		19.57		2.03		27.40	
1.63		19.75		2.04		27.60	
1.64		19.93		2.05		27.81	
1.65		20.11		2.06		28.02	
1.66		20.29		2.07		28.23	
1.67		20.47		2.08		28.44	
1.68		20.65		2.09		28.65	
1.69		20.83		2.10	26.14	28.86	10.41
1.70	19.63	21.01	7.03				

in the logarithmic curves, it is recommended that, for weirs more than 3 ft. in length, the equation, $3.242\ lh^{1.46}$ be used; for the smaller weirs, the tables give the desired results.

As an alternative, Table 19 contains the values of C for various gauge heights and sizes of weirs. By means of these values, discharge tables may be prepared for any size of weir up to 19 ft. in length.

In the foregoing experiments, no attempt has been made to develop a formula that will apply to all weirs, but merely to obtain ratings, as far as possible, and a formula or table that may be used with a reasonable degree of accuracy for large weirs with heads higher than previously tested. In this manner sufficiently accurate tables can be made and applied to irrigation work.

TABLE 15.—Gauge Height and Values of C in the Formula,

$$Q = Clh^{\frac{3}{2}}$$

Rectangular Weirs.

Series No.	Run No.	Average gauge height.	C.
6-Inch Weir.			
4	10	0.030	3.462
	9	0.042	3.256
	8	0.082	3.319
	30	0.110	3.288
	2	0.112	3.307
	1	0.117	3.192
	31	0.154	3.174
	3	0.156	3.247
	7	0.170	3.081
	32	0.208	3.143
	29	0.231	3.009
	4	0.236	3.139
	43	0.268	3.112
	28	0.309	3.050
	42	0.328	3.119
	27	0.346	3.065
	5	0.348	3.147
	41	0.398	3.106
	26	0.405	3.088
	40	0.442	3.110
	25	0.446	3.088
	36	0.472	3.114
	24	0.487	3.108
	39	0.498	3.090
	6	0.499	3.132
	34	0.524	3.127
	23	0.564	3.083
	22	0.572	3.102
	12	0.574	3.113
	33	0.604	3.119
	38	0.638	3.132
	35	0.661	3.145
	37	0.662	3.130
	11	0.678	3.128
	21	0.682	3.125
	16	0.751	3.135
	13	0.774	3.142
	17	0.803	3.141
	14	0.866	3.132
	18	0.902	3.163
	19	0.955	3.157
	15	0.992	3.168
	20	0.999	3.169
1-Foot Weir.			
13	16	0.056	3.383
	18	0.102	3.129
	17	0.142	3.327
	14	0.189	3.248
	15	0.324	3.178
	13	0.416	3.157
	12	0.518	3.138
	10	0.571	3.146
	11	0.714	3.124
	9	0.775	3.122
	19	0.822	3.131
1-Foot Weir.—(*Continued.*)			
13	8	0.857	3.130
	7	0.909	3.109
	6	0.982	3.108
	5	1.044	3.124
	2	1.126	3.133
	4	1.130	3.128
	3	1.205	3.158
	20	1.233	3.135
	1	1.338	3.147
2-Foot Weir.			
14	19	0.059	3.331
	1	0.081	3.500
	2	0.104	3.438
	3	0.167	3.382
	20	0.214	3.318
	4	0.233	3.324
	5	0.294	3.278
	6	0.395	3.256
	7	0.510	3.243
	15	0.549	3.176
	8	0.573	3.207
	9	0.717	3.186
	16	0.886	3.141
	10	0.910	3.162
	11	1.102	3.152
	12	......	......
	17	1.185	3.138
	13	1.283	3.124
	14	1.429	3.094
	18	1.574	3.130
3-Foot Weir.			
15	6	0.101	3.531
	8	0.198	3.398
	7	0.310	3.308
	4	0.385	3.367
	5	......	......
	3	0.426	3.337
	9	0.452	3.291
	2	0.468	3.333
	1	0.482	3.322
	10	0.614	3.262
	21	0.690	3.255
	12	0.826	3.257
	11	0.950	3.244
	13	1.190	3.221
	19	1.382	3.198
	18	1.401	3.185
	17	1.606	3.167
	20	1.609	3.169
	16	1.820	3.145
	14	2.058	3.160
	15	2.061	3.157

TABLE 16.—EQUATIONS AND LOCATION OF BREAKS IN LOGARITHMIC CURVES.

RECTANGULAR WEIRS.

Equation.	Gauge height of break.	Gauge height of break divided by length of weir crest.
6-INCH WEIR.		
$2.958\ lh^{1.45}$	......	......
$3.134\ lh^{1.50}$	0.328	0.656
$3.170\ lh^{1.52}$	0.572	1.144
1-FOOT WEIR.		
$3.084\ lh^{1.47}$	......	......
$3.123\ lh^{1.49}$	0.540	0.540
$3.126\ lh^{1.51}$	0.982	0.982
2-FOOT WEIR.		
$3.048\ lh^{1.45}$	......	......
$3.146\ lh^{1.46}$	0.214	0.107
$3.147\ lh^{1.47}$	0.886	0.443
3-FOOT WEIR.		
$3.242\ lh^{1.46}$	......	......

TABLE 17.—GAUGE HEIGHTS AND DISCHARGE FOR RECTANGULAR WEIRS.

Gauge height, in feet.	Discharge of 6-in. weir.	Discharge of 1-ft. weir.	Discharge of 2-ft. weir.	Discharge of 3-ft. weir.
0.00	0.000	0.000	0.00	0.00
0.05	0.018	0.037	0.08	0.13
0.10	0.051	0.103	0.21	0.33
0.15	0.093	0.188	0.38	0.60
0.20	0.142	0.287	0.58	0.91
0.25	0.195	0.398	0.83	1.26
0.30	0.254	0.521	1.08	1.65
0.35	0.318	0.658	1.35	2.08
0.40	0.391	0.801	1.65	2.53
0.45	0.469	0.950	1.95	3.00
0.50	0.551	1.111	2.27	3.50
0.55	0.636	1.282	2.62	4.01
0.60	0.725	1.459	2.98	4.57
0.65	0.818	1.640	3.34	5.13
0.70	0.916	1.830	3.74	5.73
0.75	1.018	2.028	4.14	6.35
0.80	1.125	2.233	4.54	7.00
0.85	1.235	2.448	4.96	7.65
0.90	1.350	2.668	5.41	8.33
0.95	1.465	2.892	5.86	9.03
1.00	1.584	3.125	6.31	9.73
1.05	......	3.362	6.76	10.45
1.10	......	3.609	7.25	11.20
1.15	......	3.860	7.75	11.95
1.20	......	4.119	8.25	12.70
1.25	......	4.387	8.75	13.47
1.30	......	4.664	9.27	14.27
1.35	......	......	9.80	15.07
1.40	......	......	10.35	15.87
1.45	......	......	10.90	16.68
1.50	......	......	11.45	17.53
1.55	......	......	12.00	18.38
1.60	......	.....	......	19.23
1.65	......	......	......	20.11
1.70	.....	......	......	21.01
1.75	......	......	......	21.91
1.80	......	......	......	22.86
1.85	......	... ..	......	23.81
1.90	......	......	......	24.80
1.95	.....	......	......	25.80
2.00	......	......	......	26.80
2.05	......	......	......	27.81
2.10	......	......	......	28.86

TABLE 18.—Gauge Heights and Values of C in the Formula, $Q = Clh^{\frac{3}{2}}$, for the Rectangular Weir.

Gauge height.	Length of Weir, in Feet. 6-in.	1-ft.	2-ft.	3-ft.
0.05	3.214	3.301	3.571	3.335
0.10	3.838	3.259	3.322	3.481
0.15	3.601	3.236	3.270	3.442
0.20	3.177	3.210	3.163	3.393
0.25	3.120	3.184	3.320	3.360
0.30	3.092	3.171	3.286	3.347
0.40	3.091	3.166	3.261	3.333
0.50	3.116	3.142	3.210	3.299
0.60	3.119	3.139	3.206	3.270
0.70	3.128	3.124	3.192	3.261
0.80	3.145	3.121	3.172	3.261
0.90	3.162	3.125	3.168	3.252
1.0	3.168	3.125	3.155	3.243
1.1	......	3.128	3.142	3.236
1.2	......	3.134	3.138	3.220
1.3	......	3.147	3.127	3.209
1.4	......	......	3.124	3.193
1.5	......	......	3.115	3.179
1.6	......	......	......	3.167
1.7	......	......	......	3.160
1.8	......	......	......	3.155
1.9	......	......	......	3.156
2.0	......	......	.....	3.159
2.1	......	......	......	3.161

TABLE 19.—Coefficients for Weirs with Two Complete End Contractions, for Use in the Formula, $Q = Clh^{\frac{3}{2}}$.

H = Head.	L = Length of Weir, in Feet. 6-in.	1-ft.	2-ft.	3-ft.	4-ft.	5-ft.	7-ft.	10-ft.	15-ft.	19-ft.
0.05	3.214	3.304	3.571	3.335	.....	.....	.....	.....	.....	.....
0.10	3.838	3.259	3.322	3.481	3.494	3.494	3.499	3.504	3.504	3.510
0.15	3.601	3.236	3.270	3.442	3.419	3.424	3.424	3.429	3.435	3.435
0.20	3.177	3.210	3.163	3.393	3.376	3.376	3.381	3.386	3.392	3.392
0.25	3.120	3.184	3.320	3.360	3.344	3.349	3.354	3.360	3.360	3.365
0.30	3.092	3.171	3.286	3.347	3.322	3.322	3.333	3.338	3.338	3.341
0.40	3.091	3.166	3.261	3.333	3.285	3.290	3.301	3.306	3.312	3.317
0.50	3.116	3.142	3.210	3.299	3.264	6.269	3.280	3.290	3.295	3.301
0.60	3.119	3.139	3.206	3.270	3.247	3.253	3.269	3.280	3.285	3.290
0.70	3.128	3.124	3.192	3.261	3.231	3.242	3.258	3.274	3.280	3.285
0.89	3.145	3.121	3.172	3.261	3.221	3.231	3.247	3.269	3.274	3.280
0.90	3.162	3.125	3.168	3.252	3.210	3.226	3.242	3.258	3.269	3.274
1.0	3.168	3.125	3.155	3.243	3.199	3.215	3.231	3.253	3.264	3.269
1.1	.....	3.128	3.142	3.236	3.189	3.205	3.226	3.242	3.258	3.264
1.2	.....	3.134	3.138	3.220	3.178	3.194	3.215	3.237	3.253	3.264
1.3	.....	3.147	3.127	3.209	3.167	3.199	3.205	3.231	3.247	3.258
1.4	.....	.....	3.124	3.193	3.156	3.178	3.199	3.221	3.242	3.258
1.5	.....	.....	3.115	3.179	3.151	3.167	3.189	3.215	3.237	3.253
1.6	.....	.....	.....	3.167	3.140	3.162	3.183	3.210	3.231	3.247
1.7	.....	.....	.....	3.160	.....	.....	3.178	3.205	3.226	3.247
1.8	.....	.....	.....	3.155	.....	.....	.....	.....	.....	.....
1.9	.....	.....	.....	3.156	.....	.....	.....	.....	.....	.....
2.0	.....	.....	.....	3.159	.....	.....	.....	.....	.....	.....
2.1	.....	.....	.....	3.161	.....	.....	.....	.....	.....	.....

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PAPERS AND DISCUSSIONS

SHEARING STRENGTH OF CONSTRUCTION JOINTS IN STEMS OF REINFORCED CONCRETE T-BEAMS, AS SHOWN BY TESTS.

By Lewis J. Johnson, M. Am. Soc. C. E., and John R. Nichols, Jun. Am. Soc. C. E.

To be Presented April 2d, 1913.

In reinforced concrete construction, the importance of lessening the cost of forms has long been recognized. A recent effort in this direction has resulted in a system of construction in which columns and stems of beams and girders are cast on the ground. These units, when hard, are hoisted into place and set in mortar or grouted. The concrete skeleton or framework thus erected supports the forms for the floor-slab, which is then cast in place. The slab rests on the beam and girder stems, and is secured to them by the customary web reinforcement—stirrups and bent-up or trussed tension rods.

In the ordinary beam and girder type of floor, the slab, besides receiving the live load and distributing it to the beams, performs the important function of acting as compression flange for the beams and girders. In the system just described, however, a horizontal construction joint exists between the slab or flange and the stem below, due to the lapse of several days between the casting of the two. This forces to the front the question whether this joint can transmit safely the horizontal shear necessary to make the slab and stem act together as a beam, as in an ordinary monolithic floor.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings,* and, when finally closed, the papers, with discussion in full, will be published in *Transactions.*

This paper is a record of tests which, among other things, seems to show clearly that, under conditions readily met in practice, the answer to this question is in the affirmative. These results are so strikingly at variance with ideas previously widely held that a full account of the conditions of the tests must be included.

These tests were undertaken under the following circumstances:

In December, 1911, a Boston company had contracts for two buildings on which it was proposed to use the unit construction. J. R. Worcester and L. J. Johnson, Members, Am. Soc. C. E., were retained as Consulting Engineers on the general design of the unit construction work. At their instance, a series of tests was undertaken by the writers, in the laboratory of the School of Engineering at Harvard University, to investigate the strength in shear of the horizontal joint between stem and floor-slab necessarily involved in this type of construction.

As work on one of the two buildings had to go on without delay, it was agreed, pending the completion of the tests, to introduce the precautionary measure of increasing the shear resistance of the construction joints by corrugating the tops of the stems in a manner similar to that shown in Fig. 1 (Types *A* and *B*) and in Fig. 1, Plate XXX. Thus work was begun. The tests soon made it clear, however, that the corrugations were unnecessary, and they were omitted in subsequent construction.

Objects of Tests.—The specific objects of the tests were three:

1.—To determine whether the type of construction going into the building, involving the corrugated joint between the stem and the slab, was safe;

2.—To determine whether it would be safe to dispense with the corrugations and use instead a smooth, that is, an ordinary, untreated joint; and

3.—To discover, if possible, how high a shearing stress such a smooth joint can resist.

Types of Test Beams.—For the tests, three types of beams, *A*, *B*, and *C*, were made.

Type *A* beams were designed by the Boston company primarily for the first and second objects stated. Accordingly, their design was similar to that of the beams going into the buildings.

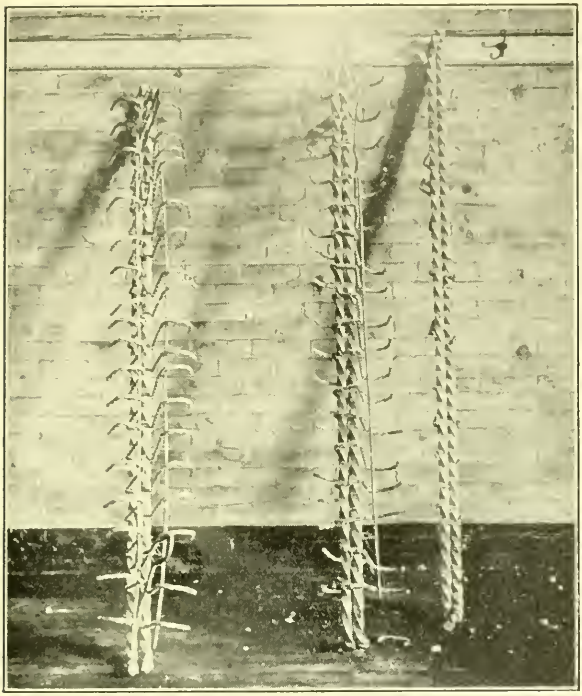


FIG. 1.—REINFORCEMENT OF THREE TYPE *B* BEAMS AFTER THE TEST. THE CUTTING OR REMOVAL OF THE STIRRUPS OCCURRED IN DISENGAGING THE STEEL FROM THE CONCRETE.

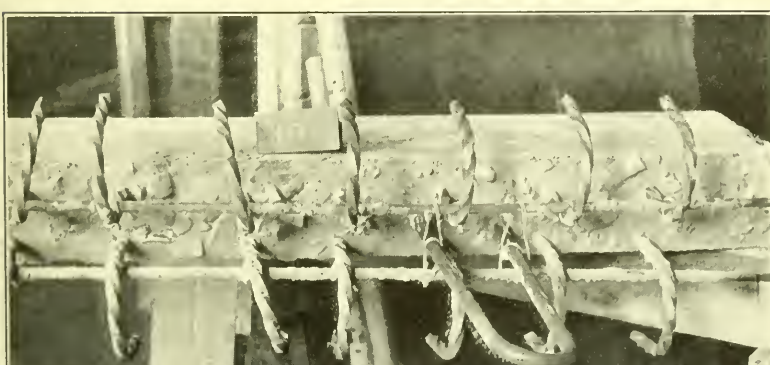


FIG. 2.—TOP SURFACE OF STEM OF BEAM NO. 16, SHOWING QUALITY OF SURFACE OF A "SMOOTH" JOINT.

Type *B* beams, designed by the authors, were intended primarily for the third object, and all the stresses were kept low in comparison with the shearing stress in the stem, in order that the shear on the joint might reach a high value before the beam failed from any other cause.

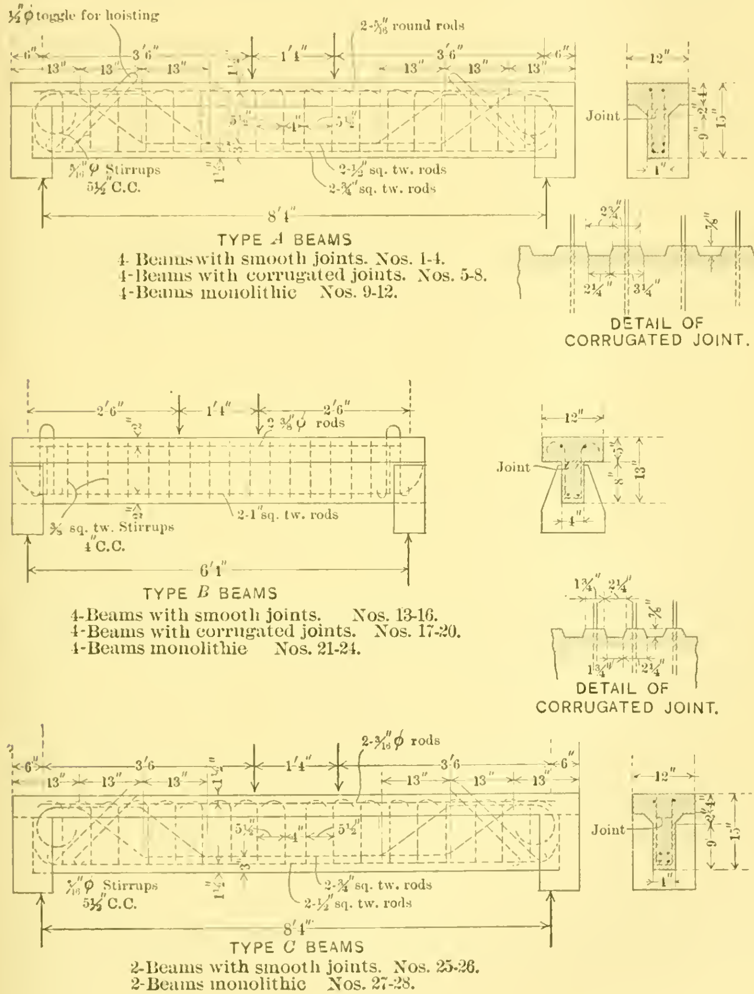


FIG. 1.

Type *C* beams, made and tested at the request of the patentees of this system of unit construction, were four in number (Nos. 25 to 28). They differed from the Type *A* beams in putting the larger

rods of the main reinforcement in the upper of the two layers and turning them up for diagonal web reinforcement, instead of having the smaller rods in this position. They differed further in having smaller rods for the stirrups. In other words, the significant difference between Types *A* and *C* is in the web reinforcement, Type *C* having more steel in the diagonal bars and less in the stirrups than Type *A*.

Twelve beams of Type *A*, numbered 1 to 12, and twelve of Type *B*, numbered 13 to 24, were made—four of each type with smooth joints, four with corrugated joints, and four cast without joints, that is, of the usual monolithic type. Of Type *C*, two beams were made with smooth joints, and two were cast monolithic.

The designs of the three types, with the essential dimensions, are fully shown in Fig. 1. The positions of the loads and supports are also shown.

Materials.—The coarse aggregate consisted of broken slate of only fair quality, dusty and somewhat scaly. In size it was 1 in. and less.

The sand was of good quality, and very similar in appearance to the Plum Island sand in common use about Boston.

Lehigh Portland cement was used. A test of the cement showed it to meet the specifications of the American Society for Testing Materials.

The main steel reinforcement was of square twisted rods, said to be cold-twisted. Measurements of the deformation of the steel during the test of the beams indicated an elastic limit in the neighborhood of from 45 000 to 50 000 lb. per sq. in. The steel for each beam was assembled and wired together in a frame before being placed in the forms.

The forms were of 2-in. plank designed so as to be easily stripped and re-assembled. Four beams or stems were cast at a time. The forms were oiled each time before the steel frame was placed.

Construction of Test Beams.—The concrete was mixed by hand in small batches, the materials being measured carefully. The mixture was what is commonly known as 1 : 2 : 4 by volume, that is, 2 cu. ft. of sand and 4 cu. ft. of stone to a bag of cement. Enough water was used to make the mass wet all through, easy to handle and spade in the forms, but not sloppy.

In order to get the main reinforcing rods into the narrow 4-in.

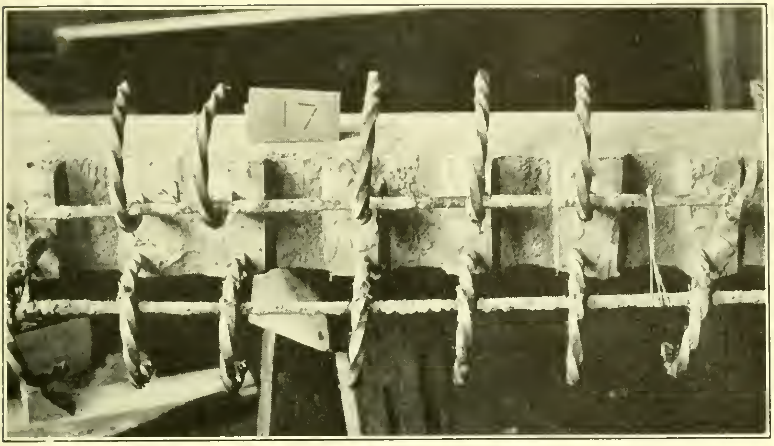


FIG. 1.—TOP OF STEM OF BEAM NO. 17, SHOWING A "CORRUGATED" JOINT.

FIG. 2.—SHOWING CRUSHING OF FLANGE BETWEEN THE LOADS, AND OPEN TENSION CRACKS.

stem, they had to be placed so close to one another (Fig. 1, Plate XXIX) that the stone could not be tamped or spaded down between them. Clear mortar, mixed 1:2, was accordingly used in the bottom of the stem, up to the top of the main rods. The concrete was placed on top of the mortar and puddled with trowels and small rods, in order to eliminate voids and secure thorough bond with the steel.

The top of the stem was leveled off in the case of the "smooth joint" beams, but not troweled. The quality of the surface is shown clearly in Fig. 2, Plate XXIX.

To make the corrugations shown in the design and in Fig. 1, Plate XXX, beveled blocks of wood were pressed down into the forms, flush with the top of the stem, and the concrete between the blocks was leveled off.

The surfaces of the joints were not subsequently brushed, or scraped, or treated in any way, except that they were thoroughly wet just before placing the concrete of the slab. The stems of the jointed beams of Types *A* and *C* were set in concrete, end-supporting blocks, in pockets, and grouted before the slab was poured. The bottom of the slab, when it was poured, came in direct contact with the tops of these blocks. The beams of Type *B*, however, were set in mortar in the concrete, end-supporting blocks, after the slab was hard. The general appearance of the three types is shown by Figs. 1, 2, and 3, Plate XXXI.

Table 1 gives the dates on which the several stems, slabs, and beams were cast.

TABLE 1.—DATES OF CASTING TEST BEAMS.

January 8, 1912	Stems 1, 2, 3, 4	(smooth).
" 9, 1912	" 5, 6, 7, 8	(corrugated).
" 10, 1912	" 13, 14, 15, 16	(smooth).
" 11, 1912	" 17, 18, 19, 20	(corrugated).
" 13, 1912	Beams 9, 10, 11, 12.	
" 17, 1912	Slabs 1, 2, 3, 4	Age of stem, 9 days.
" 18, 1912	" 5, 6, 7, 8	" " " 9 "
" 19, 1912	" 13, 14, 15, 16	" " " 9 "
" 21, 1912	" 17, 18, 19, 20	" " " 10 "
" 23, 1912	Beams 21, 22, 23, 24.	
" 25, 1912	Stems 25, 26	(smooth).
" 27, 1912	Beams 27, 28.	
" 29, 1912	Slabs 25, 26	Age of stem, 4 days.

The forms were usually removed in from 20 to 24 hours. The beams were stored in the basement where they were made, and no special provision was made for keeping them wet.

The workmanship was of commercial grade, perhaps somewhat, though not significantly, better than is secured in most reinforced concrete work. The workmen were carpenters experienced in concrete work, one of them a general foreman for reinforced concrete building construction. Although no laboratory refinements were used in the manufacture of the test beams, the workmen knew the purpose for which they were being made, and, accordingly, were careful. On the other hand, they were pushed to a time schedule of four beams a day, which required the utmost speed of which they were capable, and which in the later stages was impossible of attainment, due to the slow hardening of the concrete. To this hurry, are due slight imperfections in workmanship which appeared later. These imperfections were of three kinds:

1.—The steel was sometimes slightly misplaced;

2.—In a few of the Type *B* beams it was observed that the slab was not exactly perpendicular to the stem; and

3.—A slight curvature of the beam in plan was noticed in one or two cases.

Testing.—The beams were tested in two groups. The first tests were made on February 22d and 23d. Eleven beams were tested at this time, which sufficed to show that the type of construction going into the building was safe, and that the joints did not need to be corrugated. The remaining beams were reserved to allow the concrete to become stronger. These were tested during the week of April 15th, except Beams 21, 26, and 28 which were tested on May 4th.

The beams were tested in the 200 000-lb., Olsen testing machine in Pierce Hall, Harvard University. The beam supports consisted of rockers, which ensured a vertical reaction at the ends of the beam in all cases except that of Beam 2. During the test of this beam, the supports rocked back to the flat portion of the rocker, giving the reaction sufficient horizontal component toward the center of the span to cause the concrete supporting block to spall off with a vertical crack above the rocker. This may have contributed somewhat to the high result with this beam, but the writers do not believe that the error involved is large, and the beam has accordingly been averaged with the rest without modification.

Between the rocker and the concrete was interposed in each case a steel plate and a cushion of $\frac{3}{8}$-in. whitewood to distribute the pressure.

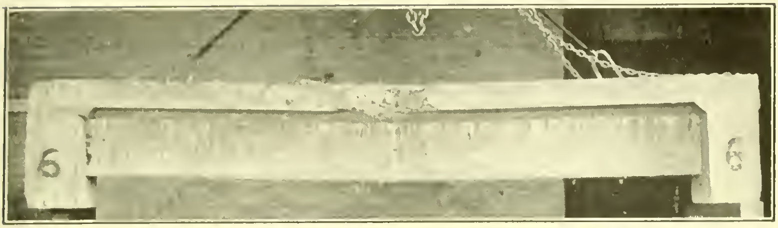


Fig. 1.—Type *A*, Beam No. 6.

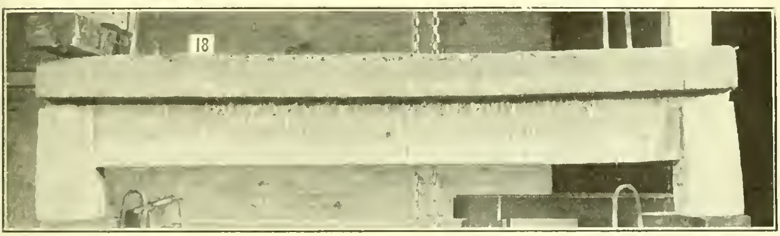


Fig. 2.—Type *B*, Beam No. 18.

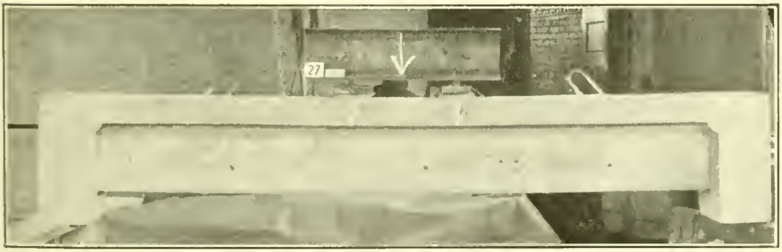


Fig. 3.—Type *C*, Beam No. 27.

The load was transmitted from the movable head of the testing machine by a pair of 8-in. I's to two steel rollers, each 8 in. from the center of the span, thence down through a steel plate and white-wood cushion to the top of the beam. This arrangement which is shown on top of beam No. 27, Fig. 3, Plate XXXI, worked satisfactorily in all cases.

The rate of application of the load was from 2 500 to 3 500 lb. per min. for beams of Types *A* and *C*, and from 5 000 to 5 500 lb. per min. for Type *B* beams, the increase in rate being due to the greater stiffness of the shorter beams. For some beams the load was removed after reaching the maximum, and re-applied at a rate, for the second and subsequent applications, some four times as fast as for the first loading.

Phenomena of Tests.—The behavior of the beams during the application of the load was remarkably uniform. Vertical cracks at the lower edge were sometimes in evidence before any load was applied, but usually appeared near the center of the span when the load was from 5 000 to 10 000 lb. These were often spaced at regular intervals of from 4 to 6 in., corresponding probably to stirrup locations.

The next characteristic event was the appearance of short fine diagonal cracks at mid-height of the stem in the region of high shear outside the loads. They were first observed at a load of from 15 000 to 20 000 lb. As no extraordinary pains had been taken to make such cracks readily visible, the actual break in the concrete may have come earlier.

As the load increased, these short cracks extended both up and down; and sometimes, but not always, joined the vertical cracks previously observed. They became at the same time more numerous and wider.

Up to this point, the behavior of the three types of beams was substantially the same. As the load approached the maximum, however, the phenomena varied. In the Type *A* beams, one or two vertical cracks developed at the center, widening and extending upward toward the fillets. Coincident with the load reaching the maximum, the top surface of the beam began to flake up between the loads, and short horizontal cracks appeared on the side of the slab, indicating crushing of the concrete. This is shown in the photograph of the center of Beam 4, Fig. 2, Plate XXX.

Beams 9 and 12 failed prematurely by spalling off the lower corner of the stem within the concrete supporting blocks, just outside of the curved ends of the lower main rods, as shown by Fig. 1, Plate XXXII.

In the Type *B* beams, the development of the diagonal cracks was the most important feature, as the load approached the maximum, until it was noticed that the main rods were slipping. In some cases the ends of the rods were uncovered and a crack between the end of the rod and the concrete was direct evidence of the slipping. Slipping was also indicated by the cracking and flaking of the concrete at the end of the beam caused by the wedging action of the twisted rods. Both kinds of evidence are presented in Fig. 2, Plate XXXII.

After the development of numerous diagonal cracks, after the rods began to slip, and after the maximum load was passed, the surface of the stem frequently began to flake off, indicating crushing in a diagonal direction. Fig. 1, Plate XXXIII, shows this phenomenon clearly.

After the maximum load had been passed with a few of the beams, the load was taken off and re-applied several times. The toughness of reinforced concrete construction is well illustrated by the high loads these beams were able to carry even when they had been badly shattered by repeated loadings.

Beams 15, 16, 17, and 23 spalled off outside the curve of the main rods in the supporting blocks, but it is probable that slipping of the rods occurred before the spalling, and was the direct cause of it.

In some of the Type *B* beams, a fine horizontal hair crack was observed at the end of the beam, level with the bottom of the slab, when the loading was well advanced. This, however, appeared in the monolithic as well as the jointed beams; it seemed to have no connection whatever with the failure of the beam, and no significance is attached to it. It was doubtless one of the local effects of the heavy concentration of the loads.

In the Type *C* beams, as the load approached the maximum, the diagonal cracks opened rapidly and the surface of the stem flaked off, followed closely by crushing of the concrete at the top of the flange at mid-span. The flaking of the stem-surface was particularly marked where the truss rods turned up, and is shown in Fig. 2, Plate XXXIII.

The numerical results of the tests are briefly set forth in Table 2.

FIG. 1.—PREMATURE FAILURE OF BEAM NO. 9 AT THE END. NOTE DEVELOPMENT OF TYPICAL DIAGONAL CRACKS. THE CHALK FIGURES ON THE BEAM INDICATE THE TOTAL LOADS, IN THOUSANDS OF POUNDS, WHEN THE CRACKS WERE FIRST OBSERVED.

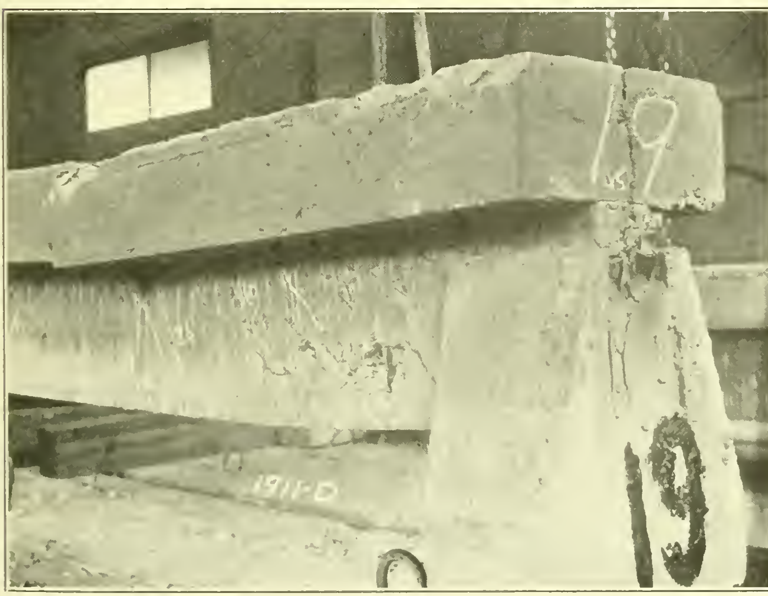


FIG. 2.—EVIDENCE THAT THE RODS SLIPPED IN AT LEAST SOME OF THE TYPE *B* BEAMS. NOTE THE OPENING ABOVE THE ENDS OF THE HOOKS OF THE REINFORCING RODS.

TABLE 2.—NUMERICAL RESULTS OF TESTS.

Type.	Joint.	Beam No.	Stem age, in days.	Slab age, in days.	Maximum load.	Computed Unit Stresses at Maximum Load. Based on design. Steel.	Concrete.	Shear.	On measurements.* Steel.	Concrete.	Shear.	Apparent immediate cause of failure.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
A	Smooth.	1	46	37	49 500	57 400	2 770	555	59 700	2 950	578	Comp. in flange.
		2	46	37	54 300	63 000	3 010	610	66 000	3 270	638	" " "
		3	101	92	50 960	59 000	2 850	572	58 600	2 820	568	" " "
		4	45	36	48 000	55 600	2 690	539	55 500	2 680	538	" " "
		Average......			50 700	58 750	2 840	569	59 950	2 930	580	
	Corrugated.	5	44	35	41 000	47 500	2 300	460	48 700	2 400	472	Comp. in flange.
		6	101	92	48 300	56 000	2 700	542	58 400	2 890	565	" " "
		7	100	91	52 100	60 500	2 920	585	61 500	2 990	594	" " "
		8	101	92	48 700	56 500	2 730	547	59 200	2 940	573	" " "
		Average......			47 500	55 100	2 660	534	56 950	2 805	551	
	Monolithic.	9	41	41	50 000	58 000	2 800	561	59 400	2 920	575	Broke in support.
		10	41	41	52 400	60 800	2 930	589	61 100	2 950	592	Comp. in flange.
		11	97	97	51 000	59 200	2 860	573	59 200	2 860	573	" " "
		12	41	41	45 800	53 100	2 560	514	53 900	2 620	522	Broke in support.
		Average......			49 800	57 800	2 790	559	58 400	2 840	566	
B	Smooth.	13	43	34	50 000	40 600	2 440	676	39 700	2 350	661	Main rods slipped.
		14	97	88	50 000	40 600	2 440	676	43 200	2 720	719	" " "
		15	98	89	55 700	45 200	2 720	754	46 100	2 820	771	" " "
		16	97	88	51 400	41 700	2 510	695	42 200	2 560	702	" " "
		Average......			51 800	42 000	2 530	700	42 800	2 610	713	
	Corrugated.	17	97	87	59 900	48 600	2 930	810	47 500	2 820	792	Main rods slipped.
		18	97	87	63 250	51 400	3 090	856	50 800	3 030	846	" " "
		19	97	87	63 600	51 600	3 110	860	53 200	3 230	888	" " "
		20	42	32	55 600	45 100	2 720	752	46 500	2 870	776	" " "
		Average......			60 600	49 200	2 960	820	49 500	3 000	826	
	Monolithic.	21	102	102	57 140	46 400	2 790	773	46 900	2 840	781	Main rods slipped.
		22	85	85	55 400	45 000	2 710	750	44 000	2 610	734	" " "
		23	87	87	55 650	45 200	2 720	752	44 700	2 670	744	" " "
		21	84	84	54 000	43 800	2 640	730	43 300	2 590	722	" " "
		Average......			55 550	45 100	2 710	751	41 700	2 680	745	

* Taking into account the actual placing of the steel.

TABLE 2.—(*Continued.*)

Type.	Joint.	Beam No.	Stem age, in days.	Slab age, in days.	Maximum load.	Computed Unit Stresses at Maximum Load.						Apparent immediate cause of failure.
						Based on design.			On measurements.*			
						Steel.	Concrete.	Shear.	Steel.	Concrete.	Shear.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
C	Smooth.	25	29	25	42 500	50 800	2 590	492	51 100	2 620	495	Comp. in flange.
		26	100	96	43 750	52 300	2 670	506	53 700	2 780	520	Comp. in stem.
		Average......			43 100	51 650	2 630	499	52 400	2 700	508	
	Monolithic.	27	27	27	45 700	54 600	2 790	529	55 000	2 830	533	Comp. in stem.
		28	98	98	47 000	56 200	2 870	544	57 700	2 990	559	" " "
		Average......			46 350	51 400	2 830	536	56 350	2 910	546	

* Taking into account the actual placing of the steel.

Discussion of Test Data.—Columns 1, 2, and 3 of Table 2 need no explanation. Columns 4 and 5 give the ages of the stem and slab, respectively, when the beam was tested. The difference between the two in any one line, gives the age of the stem when the slab was cast. Column 6 gives the total load carried by the beam, exclusive of its own weight.

The unit stresses shown in Columns 7 to 12 are computed from the maximum load, as given, on the assumptions ordinarily used in the design of reinforced concrete beams. These include the assumption of straight line or planar distribution of stress in the concrete, and a value of 15 for the ratio of modulus of elasticity of steel to that of concrete. Columns 7, 8, and 9 are based on a beam cross-section in accordance with the design. It was observed, however, that the position of the main rods varied somewhat from that intended. The actual position of these rods was carefully ascertained, and the stresses on the actual cross-section thus determined are given in Columns 10, 11, and 12.

The columns headed "Steel" record the unit tension in the main rods. Those headed "Concrete" give the maximum unit compression in the concrete at the top of the slab. The columns headed "Shear" give

the unit horizontal and vertical shear in the stem. This is also the intensity of the shear on the joint between the stem and the slab (in those beams which have a joint), the strength of which it is the purpose of these tests to investigate, and the intensity of the diagonal tension in the web, as ordinarily computed.

From the unit shear may be obtained, if it is desired, the unit adhesion on the two straight rods by multiplying the shear for Type *A* beams by $\frac{2}{3}$, for Type *B* beams by $\frac{1}{2}$, and for Type *C* beams by 1. It will be observed that both the adhesion and the shear ran high, the former to 559 lb. per sq. in., the latter to 888 lb. per sq. in.

Cause of Failure.—Type A.—The last column of the table gives what appeared to be the immediate cause of failure. It is often difficult to determine with certainty the one feature in the design of a beam which would most need strengthening in order to increase the strength of the beam. While most of the failures of Type *A* beams are ascribed to crushing of the concrete, for the reason that the crushing was the immediate visible forerunner of the maximum load attainable in the testing machine, yet it seems certain that in all cases the elastic limit of the steel was overstepped, leading to a rising neutral axis, a diminishing compression area, and a consequent premature crushing. To what extent the stretch of the steel was responsible for failure cannot be told, but it probably was the prime cause in all cases. It is not essential to this investigation to know.

Two of the Type *A* beams failed in the supports, as noted, by spalling off the lower corner of the stem below and outside of the curve of the lower rods.

Cause of Failure.—Type B.—The primary cause of failure of the Type *B* beams was probably slipping of the main tension rods, although this was not observed in all cases before the maximum load was attained. The further destruction of the stem and slab by cracking and crushing under continued application of the load, cannot be regarded as the cause of failure. There is every probability that, if the same amount of steel had been secured with a larger number of smaller rods, the strength of all the Type *B* beams would have been materially increased.

Cause of Failure.—Type C.—In Beam 25 it was not observed whether the maximum load was passed before the crushing of the flange began. The failure appeared to be due to this crushing, but the stem

was also badly cracked and had begun to crush diagonally, especially in the lower turn of the truss rods. Furthermore, the elastic limit of the main steel was doubtless overstepped.

In the case of the other three beams of Type *C*, the maximum load had clearly been reached before the flange began to crush, but the crushing of the stem was coincident with or preceded the maximum load. How far the failure of the concrete in bearing at the turn of the truss rods, and the consequent straightening and yielding of the rods themselves, reduced the effectiveness of the web reinforcement, and thus became the primary cause of failure, cannot be told. Neither can the effect of passing the elastic limit of the main rods be estimated.

Further evidence that the steel was the limiting factor in the strength of the beams lies in the fact that there is only a very slight difference in strength between beams 40 days old and those 90 days old. If the concrete were the primary cause of failure, this difference should have been more marked.

General Results.—Horizontal shear or slipping on the joint was looked for in all the beams with great care, but was never observed. In fact, the jointed beams, both smooth and corrugated, withstood the test as well as the monolithic. There was not the slightest evidence that the joint contributed in any way to the failure of the beams, though the shearing stress on the joint reached values greatly in excess of four times the 120 lb. per sq. in. allowed by the Joint Committee in beams reinforced for shear.

Three interesting load-deflection curves, made by an autographic device on the testing machine, are presented in Fig. 2.

Conclusions.—In view of the striking uniformity, both in the general behavior of the beams during the test and in the numerical results attained, these tests would seem to justify the following conclusions:

1.—That the type of construction used on the buildings was safe;

2.—That the joints did not need to be corrugated; and

3.—That a smooth or unroughened joint, constructed in the same manner as the joints tested in a beam suitably reinforced for shear (or diagonal tension), is capable of transmitting with ample factor of safety any shear that could safely be permitted in a monolithic beam of the same cross-section and having the same reinforcement.

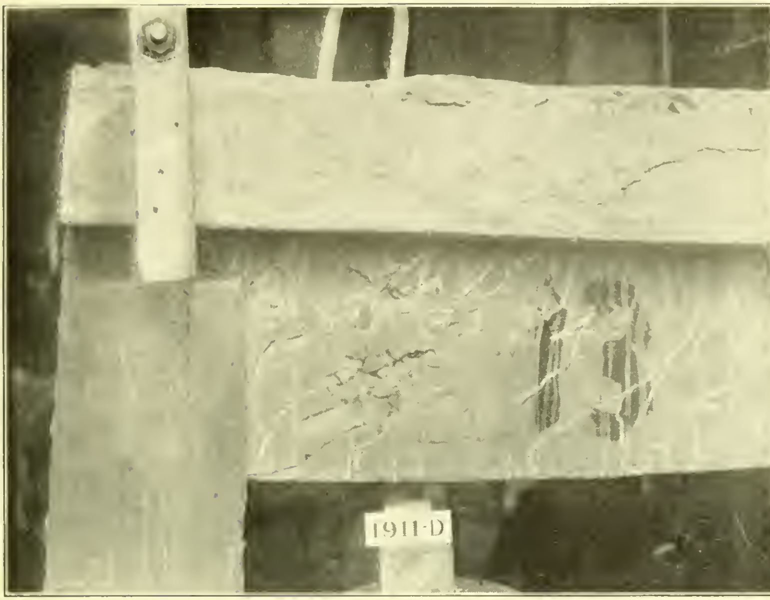


FIG. 1.—DIAGONAL CRUSHING OF STEM UNDER LOADING CONTINUED AFTER MAXIMUM LOAD HAS BEEN PASSED.

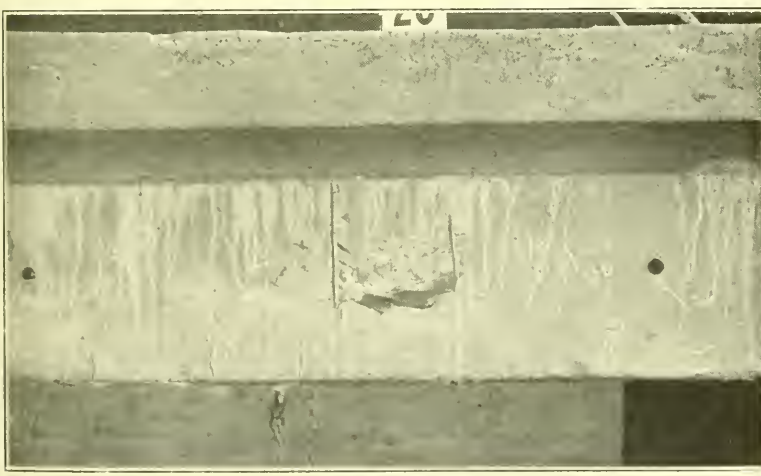
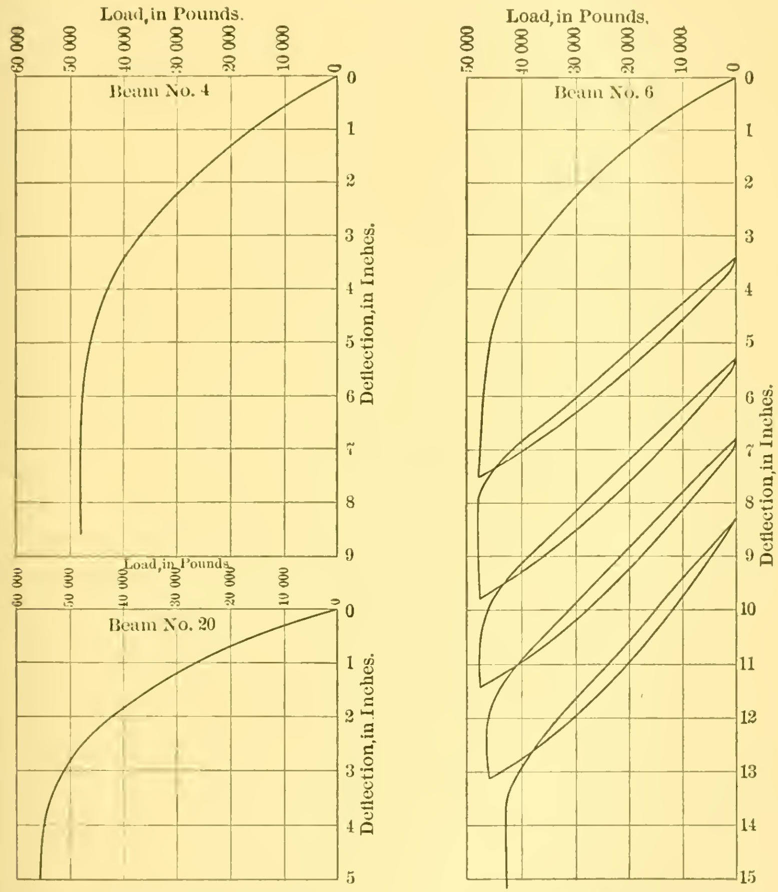


FIG. 2.—CRUSHING OF CONCRETE WITHIN TURN OF BENT-UP ROD IN BEAM NO. 26.

One Important Bearing on Engineering Practice.—It may reasonably be inferred from the results of these tests that in the ordinary monolithic floor construction, where adequate web reinforcement is provided with sufficient anchorage at the top, the slab need not be



Two Typical Load-deflection Curves. Note the Greater Stiffness of the Type *B* Beam, No. 20.

Load-deflection Curve for Beam No. 6. Load Removed and Reapplied Several Times. See also Fig. 2, Plate XXXI.

FIG. 2.

cast before the stems have set, although the contrary has hitherto been generally assumed. Until further tests are made, however, care should be taken to see that the web reinforcement crossing the joint is sufficiently anchored, both above and below, and the surface of the

joint must be clean and not too smooth. Since these are conditions not difficult to obtain in practice, there may be now in sight an escape from the worries caused by the occasional necessity of stopping work when the forms are filled only to the bottom of the slab.

Allowable Shearing Stress.—Furthermore, it appears that the stem of a reinforced concrete T-beam may be reinforced so as to be capable of carrying an ultimate shearing stress of at least 888 lb. per sq. in., the highest reached with any of the beams. As the shear was in no case clearly the cause of failure, the really ultimate shearing strength was probably not attained. How much above 888 lb. per sq. in. it really is cannot be stated. The high values of unit shear reached in these tests, together with the fact that the lowest value for the unit shear in any of the beams when the first diagonal crack was observed was in the neighborhood of 185 lb. per sq. in., indicate that the 120 lb. per sq. in. of the Joint Committee's recommendation is low enough, provided the shear reinforcement is adequate. It should be remembered, too, that the concrete of these tests was not of exceptional richness, nor otherwise better than ordinary building quality.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

FREMANTLE GRAVING DOCK: STEEL DAM CONSTRUCTION FOR NORTH WALL.

By Joshua Fielden Ramsbotham, Assoc. M. Am. Soc. C. E.

To be Presented April 2d, 1913.

Toward the end of 1910, the writer was appointed by the Western Australian Government to design and carry out departmentally the Fremantle Graving Dock, the construction of which was in hand under the Engineer-in-Chief of the Public Works Department, Mr. James Thompson. When the writer took charge, an area had been dredged for the dock to a depth of 49 ft. below low-water mark, the ordinary rise of tide being about 3 ft. 6 in. and extraordinary tides rising only 6 ft. The tidal rise and fall are irregular and intermittent, and, to a large extent, depend on the direction of the wind.

The whole policy of future work depended on the core borings, which had been taken previous to the writer's appointment. From these 73 core borings (two typical bores are shown on Fig. 2) it will be noted that the ground consisted generally of a calcareous sandstone containing pockets of sand, with a bed of clay at a lower level.

After careful consideration and examination of the dredged spoil, the writer advised the Engineer-in-Chief to recommend the Government to abandon the site. From the plan, Fig. 1, it will be noted that by doing this, the entrance and fairway to the harbor would be con-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

siderably improved, and the work already done could be turned to advantage.

However, costly enterprises and large undertakings are not to be changed lightly, and it was decided to put down a shaft, make an examination of the bottom, and thoroughly test the porosity of the ground. For this purpose a shaft, 8 ft. 6 in. square, was built, consisting of 12-in. steel piles, 60 ft. long, manufactured by the United States Steel Products Company, the minimum head being 48 ft.

This shaft or dam was unwatered successfully by two 7-in. pulsometer pumps, no puddled clay being used on the outside. On pumping down to 26 ft. below low water, a burst or "blow" occurred under the root of the piles, which, to a large extent, choked itself; but it was sufficiently sealed finally by a "swab" of Italian gasket attached to a sinker, which enabled the shaft to be pumped out. On examination it was found that, at 48 ft. below low water, the rock was hard on the north side, but very soft in the remainder of the area, and it had been noticed that three of the piles had refused to drive home on the north side, while the remainder drove easily, a few so easily (1½ in. at a blow) that, although they were not driven to a set, driving had to be stopped or they would have been under water.

This variation of ground in such a small area was very marked, and, in his report, the writer pointed out that it was problematical whether it would be safe to found a dock in this particular locality, and, further, that there were indications that heavy pumping would be necessary.

On drawing the piles it was noticed that they had penetrated a bed of clay, thus confirming the borings, and also showing that the clay was of good quality.

The writer again advised the Government, on the grounds of the uncertainty of the bottom and of the initial cost, to change the site. It was decided finally by the Government to curtail the sphere of operations to the north wall of the dock, and, in the event of the ground being found unsatisfactory and the ultimate cost prohibitive, to abandon the dock and construct a quay for commerce. At the same time the spit of rock was to be dredged out, thereby considerably improving the harbor.

In work of this character prime cost is very often not sufficiently realized, but in this case estimates were prepared, from data obtained


DETAIL OF BORES
INDIAN OCEAN
NORTH FREMANTLE
FREMANTLE
Rocky Bay
Freshwater Bay
Buckland Hill
Obelisk
Signal Hill
FREMANTLE GRAVING DOCK
PROPOSED SITES
SCALE OF FEET
1000 0 1000 2000 3000
Note
Proposed Graving Dock Sites shown thus
Floating

FIG. 1.

from similar work, accounting for every item except pumping, which, considering that it was deemed impossible to ascertain, was left as an unknown quantity, with the intimation that the cost would be excessive. From the borings it will be seen that the material in which the piles were driven consisted of a hard limestone cap, coral, clay, and sandstone, the limestone cap being particularly hard in a portion of the pumping station, where a total of 60 ft. of driving had to be done, or, in other words, each pile had to be driven 60 ft. into the ground.

Public tenders were called for, and the lowest, that of the United States Steel Products Company, was accepted.

It is worthy of record that, in driving one of the steel piles, a 45-lb. per yd. steel rail was cut into two pieces, one section being found subsequently on excavating within the dam.

From a driving point of view, these piles are admirable, as the cross-sectional area, or displacement, is small compared with the covering area of the face of the pile, it being very much like driving a knife into a pat of butter, and yet, although whippy, with care very few piles were damaged; and, further, in shipping 1 900 piles, 60 ft. long, from the United States to Western Australia, not one was damaged, and only two were damaged in handling for driving. In handling from the steamer it was found best to have 8 by 3½-in. fish-plates of Oregon pine, 10 ft. long, with a sling chain around them, placed 20 ft. from one end.

In handling, preparatory to driving, a single piece of 7 by 3-in. Oregon pine, 8 ft. long, was found best, and was slung one-third from one end.

Design of Dam.—The design of the dam, to a large extent, was assisted by the results attained in the small trial shaft. The object was to have as few sets of timber as possible, consistent with safety, and as all the shoring had to be done by divers, both from the point of view of economy and speed, this was most important.

The points that require careful watching in designing a dam are:

1.—Strength of skin (which can be taken as a beam supported at several points, and to an extent this is advantageous, as benefit is derived from contraflexure);

2.—Strength of walings or longitudinal beams;

3.—Strength of shores or transverse columns; and,

4.—Factor of safety on the dam.

VIEW OF DAM FOR FREMANTLE GRAVING DOCK.

Great benefit may be derived from working in water, as it is possible to calculate exactly the loads which the various members of the structure have to carry; and, needless to say, the object is to place the walings and shores so that they and also the skin will have an ample factor of safety.

For many reasons, it was decided to use jarrah for the walings and shores, and, as the writer had no previous knowledge of this timber, it was with no small interest that he watched its behavior. As jarrah is a product of Western Australia, a supply could be obtained at the reasonable price of 2 shillings per cu. ft. delivered in railway trucks on the job. It is nearly always heavier than water, and consequently will sink, which is an advantage for divers' work; its strength lies in its outer fibers, therefore it is safer to use hewn than sawn jarrah, as hewers will always hew with the grain.

As heart timber was being accepted in this case, the heart was in the neutral axis or center of the log; further, as jarrah is short in grain, it is certainly advisable not to accept sawn logs, as there is danger of the grain running out unless great care is exercised. In addition to this, as the dam was being built in sections, the divers keeping ahead of the pumped-out sections, the *Teredo navalis* had to be guarded against, and jarrah is the best local timber for resisting this pest.

In designing the dam, a maximum tide of 6 ft. 6 in. above low-water mark was considered (this is seldom if ever reached), and the dredged bottom was taken at 49 ft. 0 in. below low-water mark, thus giving a total calculated head of 55 ft. 6 in., which in actual work could be taken at 51 ft. Further, clay puddle was tipped to a minimum height of 9 ft. at the skin of the dam, in order to make a seal of the porous rock, and, at the same time, reduce considerably the head on the dam. Assuming that the shores are placed at 10-ft. centers horizontally, the pressure on a strip of dam, 10 ft. wide, is:

$$\frac{55 \text{ ft. } 6 \text{ in.} \times 10 \times 55 \text{ ft. } 6 \text{ in.} \times 64}{2 \times 2\,240} = 440 \text{ tons.}$$

The strength of a jarrah beam, loaded uniformly and supported at each end, is given by the following formula, in which K is a constant which has been ascertained to be 13; B is the breadth, and is

assumed to be 14 in.; D is the depth, and is assumed to be 14 in.; and L is the length, which is 9 ft., or 108 in.:

$$\text{Breaking weight, in hundredweights} = \frac{8\,K\,B\,D^2}{L}$$

$$= \frac{8 \times 13 \times 14 \times 14 \times 14}{108}$$

$$= 2\,642 \text{ cwt.}$$

$$= 132 \text{ tons.}$$

Thus, using a factor of safety of 3, the safe load $= \frac{132}{3} = 44$ tons.

From this the number of walings or beams required is $\frac{440}{44} = 10$.

It is worthy of note that, in calculating the strength of walings, no allowance was made for the fact that the beams were continuous over several supports, thus giving a greater margin of safety than is shown. The shores or columns were 12 in. square and 22 ft. 8 in. long, and were worked out by Gordon's formula in a similar manner, one end being considered rounded and the other fixed, the factor of safety being slightly less than 3.

When the driving of the piles was completed, it was found that it was impracticable to keep them absolutely vertical or in line, due to the hard nature of the ground; and, although the divers packed between all piles and the back of the walings, a certain amount of unquestionable bridging or arching took place. This was so marked that, without any further delay, additional shores were placed alongside those already in, the factor of safety of the walings thus being increased to $3\frac{3}{4}$ and the shores to almost $4\frac{1}{2}$.

The skin of the dam is important, and the largest span was settled empirically from observations taken while pumping out the small trial shaft, and was a clear span of 8 ft. 8 in., subjected to an actual head of 17 ft. 7 in. of water. In the dam in question, the span was 8 ft. 7 in. between the second and third shores. Between the first and second shores the clear span is 9 ft. 11 in., and, although allowance was made for a high tide of 6 ft. 6 in., it is seldom if ever reached. From the writer's experience, a clear span of 8 ft. 8 in., subjected to a mean head of 16 ft. 8 in., is quite satisfactory, the total load on a pile being 4.88 tons, and the maximum stress about 10 tons per sq. in. From the foregoing it may be noted that uniformity has been reached in the temporary dam, and certainly not at the expense of safety.

The embayment for the pumping station offered difficulties outside the ordinary section of the dam, and the writer proposes to touch on it briefly at this stage.

The maximum length of the shores (shown on Fig. 2) is 57 ft. They were built up of 14 by 14-in. jarrah logs, with fish-plates at top and bottom consisting of 20-ft. half timbers of jarrah (14 by 7 in.), without heart, held together with fourteen 1-in. bolts. Care was taken to alter the lengths of the timbers in each line of shores so as to alter the position of the joint; this is important.

In turn, these shores were subdivided, by vertical and horizontal "toms", into small shores 18 ft. in length. Each tier of shores was carefully anchored to the line below by long bolts, and, during excavating, they were invariably kept packed down to the ground by relieving "toms". Further, a deflection of 1½ in. in the center was allowed so as to overcome any tendency of the shore to bend upward or break, and, in addition, there was always considerable weight on the top setting, due to the staging carrying the crane and railway tracks. No trouble was experienced in any way from these shores, the factor of safety on them being 5.27.

Practical Points on Construction.—Important points were observed during the construction, and perhaps the writer may be pardoned for stating them in some detail.

First, it is of the utmost importance that the female end of the pile should always be driven over the male, and not the male in the female. If the latter is done, a column of spoil will be compressed under the male in the jaws of the female which will finally end in the bursting of the structure. To enable rapid progress two piles were split down the center and a double male pile manufactured, riveting a 6 by 3 by ½ in. T-iron on the back for stiffness. By this means two piling machines were utilized; one was built on an outrigger, with one end supported on staging and the other end on a log on the ground, the piling frame, winch, boiler, etc., being self-contained and easily moved. The other machine was on a barge, and was also self-contained.

In passing, it may be mentioned that the piling winches were made by the Lidgerwood Manufacturing Company (No. 72¼), and were capable of working a 2-ton "tup" or hammer. A 30-cwt. tup was found to be best in practice, and a good man can get 20 blows per

min., a 5-ft. drop being quite sufficient. From the plan it will be noticed that provisions were made for subdividing the dam every 100 ft. by using T-piles, a male and female T-pile being left opposite each other; this was done in accordance with the catalogue of the makers of the piling. When the time came to drive, however, difficulties arose, and as they could be overcome very easily by the manufacturers of the piles, the writer mentions them, hoping that he may thus assist others.

First.—By having a female T-pile, it would mean, in the ordinary course, driving a male pile in it; as previously mentioned, if this is done, it will end in disaster, unless the spoil in which it is driven is a fluid mud. To overcome this, a double female pile was made, and on one side a strip of iron, 1 in. wide and ½ in. thick, was tapped on the outside on one jaw in order to make a lock. The jaws were then interlocked, and the pile was driven, there being clearance for any displacement of spoil between the jaws.

Second.—Great care must be exercised in keeping the two T-piles parallel, otherwise obvious difficulties will arise in closing. The manufacturers could overcome this by making an expanding pile with slotted holes, so as to enable the closing pile to vary its width from the top to the bottom.

Third.—Even with parallel piles, it is extremely hard to marry in with the closing pile, and it is desirable to have a few piles of different width.

Fourth.—If a male T-pile is used at each side of the dam, as should be done, a double female pile is required with which to end, and such should be supplied by the manufacturers.

Fifth.—If a male T-pile is urgently required, it can easily be made, and at small cost, by riveting an ordinary 45-lb. rail to an ordinary pile.

Sixth.—In driving the piles, the moment one seems to be hardening up and has not reached the required depth, another should be driven ahead of it, and, if necessary, this should be repeated until there are signs of having reached soft ground again. It is then possible to exert more force, but even then abuse must not be resorted to, as there is grave danger of parting the piles, but if this does happen by having driven ahead, that danger is limited in extent and is only local.

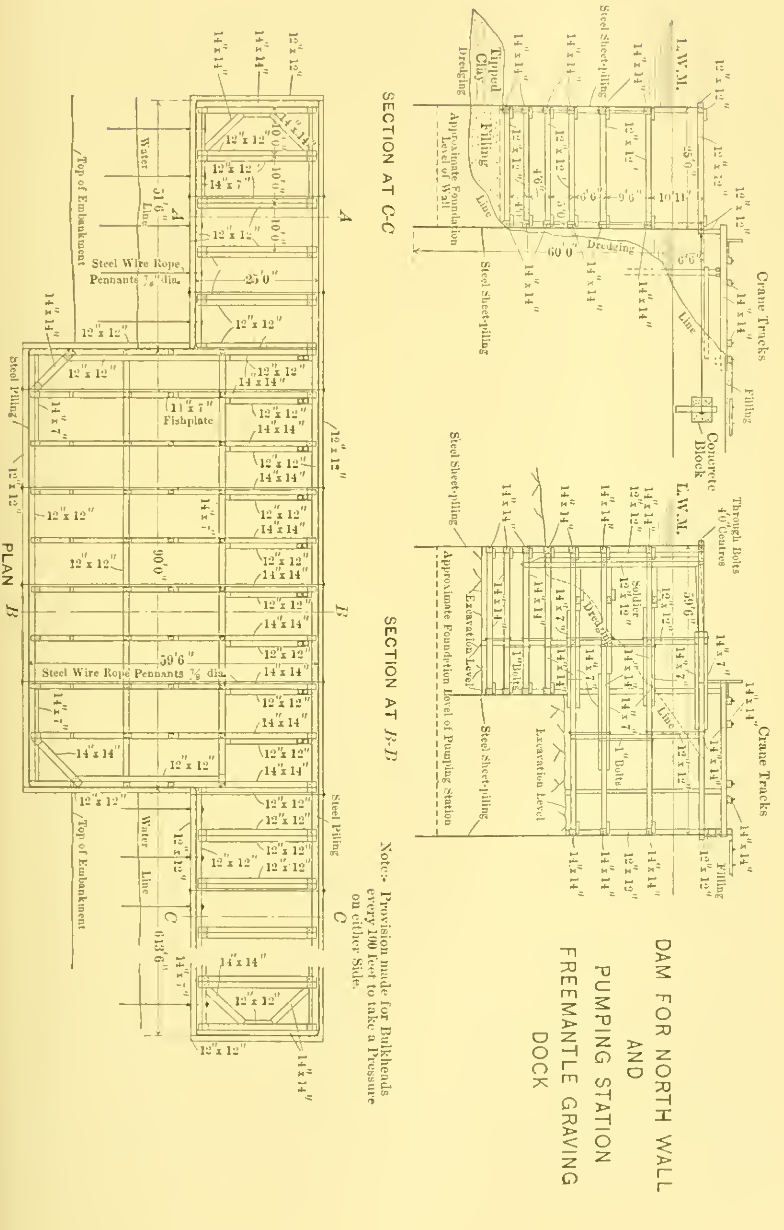

DAM FOR NORTH WALL AND PUMPING STATION FREMANTLE GRAVING DOCK
SECTION AT C-C
SECTION AT B-B
PLAN
Note:- Provision made for Bulkheads every 100 feet to take a Pressure on either Side.

Fig. 2.

From the cross-section it will be noted that the piles toward the shore were driven on the side of a cliff, which had been left in a very irregular condition by the dredging, and added considerably to the difficulties of the piling gang; it also rather retarded the progress, as some piles had to be drawn. A fair month's work for the two machines, there being two 8-hour shifts on each machine, averaged 370 piles, representing 9 361 lin. ft. of driving.

The first steel pile was driven on July 14th, 1911, and pumping was started in Dam Section No. 1 on November 16th, the entire dam being framed by the end of that year.

It is interesting to note that 1 872 steel piles were used, representing 47 385 lin. ft., or 9 miles of actual driving. In the first dam section there were 401 piles, which gives 24 060 lin. ft. of joint, representing very nearly 5 miles of joint.

Constructional Work.—At the top of the piles it will be noted that a substantial back-strap is provided, consisting of a 12 by 12-in. jarrah log. This strap was bolted through every fourth steel pile and the inside waling by a 1-in. bolt. Although no trouble was experienced, the writer would recommend $1\frac{1}{4}$-in. bolts for this purpose.

The walings were hung by two 1-in. bolts from the top timber, breaking joint, and between each, there were placed distance pieces or "toms", cut to the required lengths; in this way the necessary accuracy of placing the walings by the divers was assured, and the progress was rapid.

Owing to the fact that the piles were over at the heads and out of plumb, there was a tendency to lift the walings, so that the "toms" and through bolts play an active part, and require most careful watching, as in the event of any small movement, a disaster of great magnitude to life might result, and almost without any warning. No movement was observed, but careful examinations were made and soundings were taken every 10 ft. daily on the clay outside the dam.

When necessary, packings were placed between the walings and the steel piles by the divers, and, when the dam was pumped dry, their work was found to have been done both creditably and satisfactorily.

Across the embayment to the pumping station some interesting work was done. As there was only a sloped bank to which to shore, it was obvious that some other method would have to be adopted. Accordingly, half timbers were bolted on the long shores, top and

FIG. 1.—EXCAVATION FOR PUMPING STATION, FREMANTLE GRAVING DOCK.

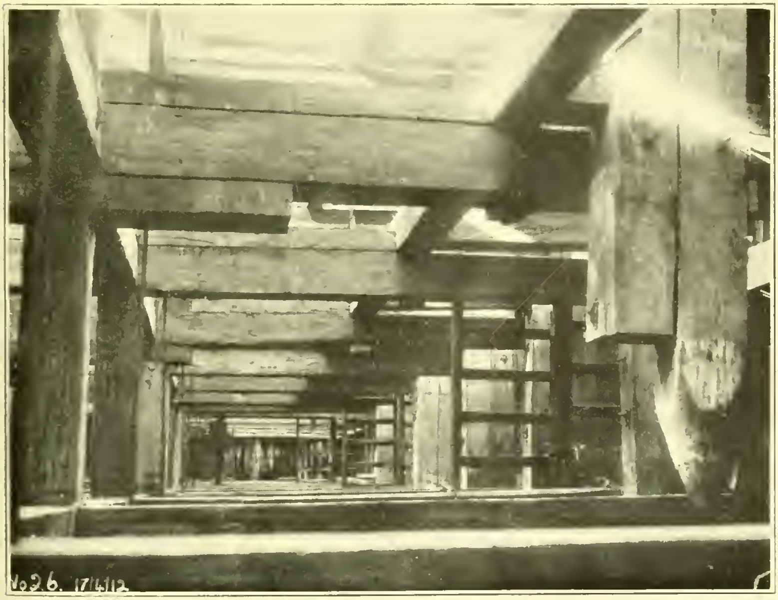


FIG. 2.—METHOD OF SHORING DAM FOR FREMANTLE GRAVING DOCK.

bottom, and packed against the top and bottom of the fish-plates which served to build up the long shores and for which purpose fourteen 1-in. bolts were used. This is shown clearly by the section at *B B*, Fig. 2.

King piles, composed of two 14 by 14-in. jarrah logs, bolted side by side, were driven into the ground, and shores were then placed by the diver from the waling to the king pile, and again shored to the bolted half timbers, as previously mentioned, when the dam was pumped down.

As soon as the water was below each setting, the long shores were put in, and all "tomming" and "lacing" done at once.

Pumping Plant.—Three 15-in., centrifugal pumps were used, each capable of working as follows:

Total head, in feet.	Revolutions, per minute.	Capacity, in gallons per minute.
40	500	4 900.
50	500	4 400.
60	500	3 800.

The weight of the pumps and engine combined was about 4 tons, so that they were within convenient compass of the 5-ton cranes in use.

Allen "Conqueror" centrifugal pumps were used, and were direct-coupled to "Allen" enclosed two-cylinder compound engines. Each engine developed 110 h.p. at 450 rev. per min. and a steam pressure of 110 lb. per sq. in., the diameter of the cylinders being 10 and 14 in., and the length of the stroke 6½ in.

The boilers were made by Ruston, Proctor and Company, Limited, of Lincoln, England, and at first consisted of two semi-portable, multi-tubular boilers, each having a grate area of 26 sq. ft., capable of evaporating 4 000 lb. of steam per hour, with a pressure of 150 lb. per sq. in., the heating surface being 814 sq. ft. Their length over all was 20 ft. 1 in., their diameter, 4 ft. 6¼ in., and each contained 105 2½-in. tubes. The weight of each boiler without fittings was 9 tons 2 cwt., and with fittings, 11½ tons.

Pumping was started on November 16th, 1911, and continued without intermission until July 23d, 1912. It was found necessary to add another boiler, due partly to the low calorific value of the coal used, but mainly on account of the necessity for cleaning and washing out the boilers, such operations being prejudicial to the continuous running

of three pumps. In addition, one 7-in. steam pulsometer pump was used for well sinking, etc.

The Imperial Electric and Engineering Company, of Perth, Western Australia, was the local agent for the above firms, and it is satisfactory to note that from start to finish there were no breakdowns, a result which is most satisfactory when the entire work is at the mercy of the pumps.

Work Done Under Cover of the Temporary Dams.—The pumps were first placed in the bays by the bulkhead, one pump taking up one bay of the dam. They were supported on a staging well braced, checked, and bolted to timber piles driven inside the dam. At first all three pumps were engaged, but gradually a reduction in the pumping took place. The area to be unwatered was 10 453 sq. ft.

After excavating some 20 ft., it was found that, due to the hardness of the rock in that locality, some of the piles had not driven. On reaching a level of 20 ft. below low-water mark, it was decided to excavate a 10 by 7½-ft. well for the temporary pumps, down to 43 ft. below low-water mark, within steel piles, and in the center of the main pumping station, as shown on Fig. 3. This was done, using a 7-in. pulsometer for sinking purposes. Sulphureted hydrogen gas was encountered, which caused very great discomfort to the workmen, some of whom became temporarily blind, others developed eczema, and every one concerned had a bad yellow color and became "as lean as crows", suffering from severe attacks of cramps and general debility. A silver coin placed in the water became of a bronze color in a few seconds. In front of the well, the piles were of steel cut into 5-ft. lengths, and a grating was provided which worked in a groove, so that, when excavating in the body of the dam, a 5-ft. length would be removed, and the pumps were protected by the grating. This answered admirably, and no inconvenience was caused by the silting up of the rose on the pumps.

Unfortunately, after everything had been completed in the shaft. a blow occurred in the bottom of the well, and, although not serious, it was necessary to discover its source. The first thought was that the flow might be coming under the ground where the piles had not driven satisfactory, or, in other words, the hydraulic gradient had not been cut off. To ascertain this, holes, 30 ft. deep, were drilled 5 ft. away from the skin of the chamber opposite the gap in the piling and 5 centers apart; on drilling to a depth of 28 ft., or 48 ft. below

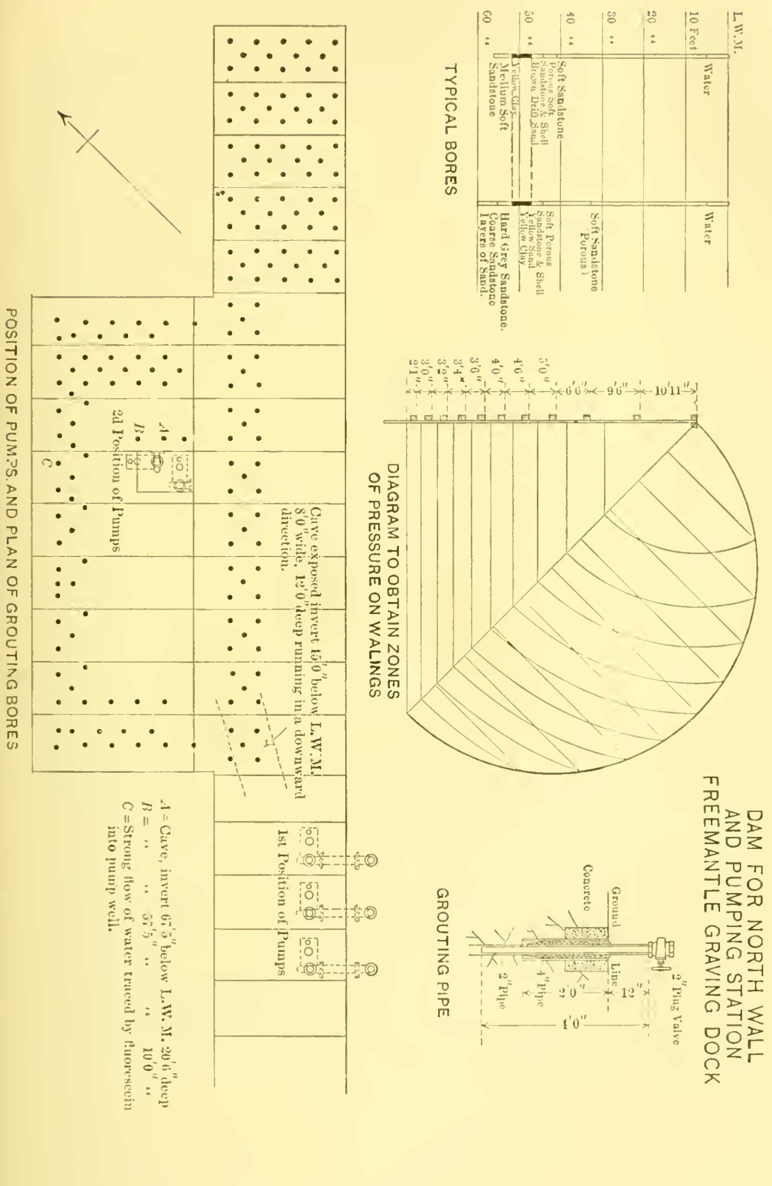

DAM FOR NORTH WALL
AND PUMPING STATION
FREEMANTLE GRAVING DOCK
TYPICAL BORES
L.W.M.
10 Feet
Water
Soft Sandstone
Porous Soft Sandstone & Shell
Brown Drift Sand
Yellow Clay
Medium Soft Sandstone
Soft Sandstone (Porous)
Soft Porous Sandstone & Shell
Yellow Sand
Yellow Clay
Hard Grey Sandstone.
Coarse Sandstone
layers of Sand.
DIAGRAM TO OBTAIN ZONES
OF PRESSURE ON WALINGS
GROUTING PIPE
2" Plug Valve
Ground
Line
Concrete
4" Pipe
2" Pipe
Cave exposed invert 15'0" below L.W.M.
8'0" wide, 12'0" deep running in a downward direction.
1st. Position of Pumps
2d Position of Pumps
A = Cave, invert 67'5" below L.W.M. 20'6" deep
C = Strong flow of water traced by fluorescein into pump well.
POSITION OF PUMPS AND PLAN OF GROUTING BORES

Fig. 3.

low-water mark, the drill dropped 2 ft. and water spurted up through the hole. It was thought that the source of the trouble had been found.

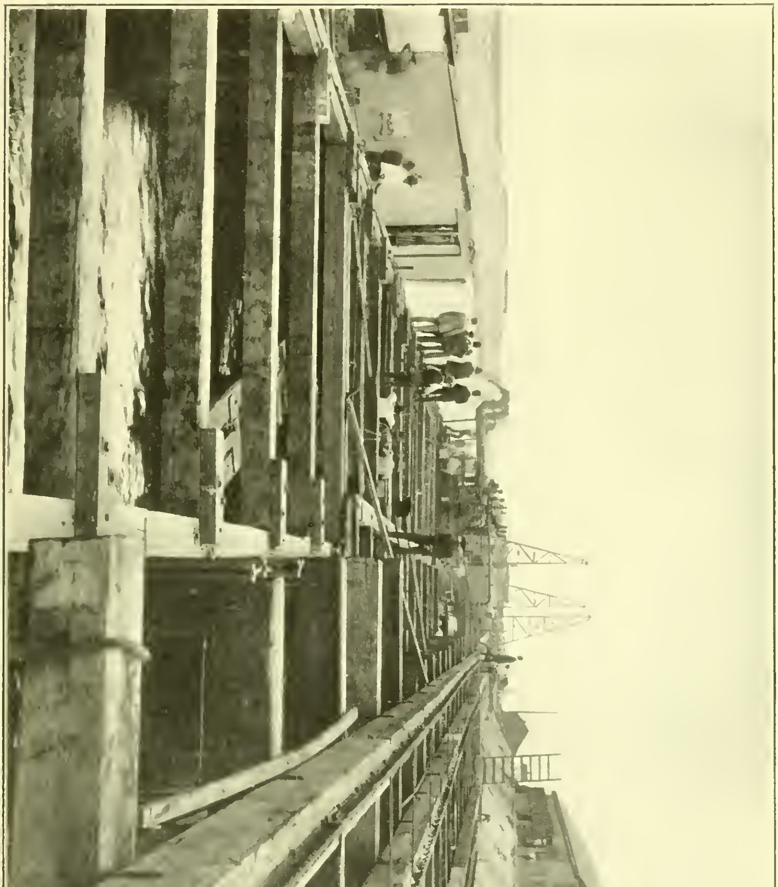
To try and trace the flow of water, a strong solution of potassium permanganate ($KMn\,O_4$) was forced under pressure down the drilled holes, but no results were observed. On mentioning the matter to Mr. E. A. Mann, the Government Analyst, he pointed out that the sulphureted hydrogen would change the purple color of the permanganate rapidly to brown, and then to a colorless solution, this being due to the sulphureted hydrogen ($H_2\,S$) changing the permanganate to an oxide of manganese with a deposition of pure sulphur. Some sulphate due to the formation of sulphuric acid ($H_2\,S\,O_4$) is also probably formed.

Mr. Mann kindly gave this question of the presence of sulphureted gas his attention, as it was very noticeable that there was considerable corrosion on the steel piling, in addition to the other features mentioned previously. The writer will refer to this matter later.

For tracing the flow of water, Mr. Mann recommended fluorescein or resorcinolphthalein ($C_2\,H_{12}\,O_5$), a few grains of which gave an extremely bright green fluorescence, which did not lose its color even in a considerable volume of water.

Almost immediately a flow of water was traced in the main dam, a few seconds only elapsing after putting in a few grains of fluorescein before its presence was apparent. Accordingly, systematic drilling and grouting under pressure was adopted, and as it was novel to the writer, he may be pardoned for referring to the subject in some detail.

Drilling.—It was found advisable, from experience, first to procure a 4-in. iron pipe, from 18 to 24 in. long, and concrete it flush with the ground with a 12-in. cube of concrete, as shown on Fig. 3. This was used for drilling purposes, and prevented the ground from being fretted by the drill; it also stopped any leakage during grouting under pressure. Machine drills and compressed air were available for drilling, but the strata were such as to render them useless, and it was found to be cheaper to jump the holes down by hand. A chisel drill, 3 in. across the bit, was used, and, in the pumping-station area, 30 ft. was the minimum depth of hole drilled. On reaching a level of 48 ft. below low-water mark, as previously mentioned, a cave 2 ft.



Dam at Commencement of Pumping.

deep was entered, and a strong flow of water under pressure was set free. The hole having been drilled to the requisite level, an iron pipe, 3 ft. long and 2 in. in diameter, was hammered in. This pipe had been previously roughened on the outside and tightly wound with strips of soft bagging, making a tapered shape very much like an ordinary top; this was well tallowed and tightly bound with strong twine; on the upper end of the pipe was a thread on which was screwed an ordinary 2-in. straight-way valve. In some cases tapered wooden plugs were used until immediately previous to grouting, when they were replaced by the pipe valve. It may be mentioned that there was no trouble from leakage, even under a pressure of 75 lb. per sq. in.

Grouting Machine.—A grouting machine, Fig. 5, similar to the Canniff tank grouting machine* was made. Small modifications, which possibly might be regarded as improvements were introduced, such as having no bends in the pressure pipes, such bends being obviated by having T's fitted with plugs and used for cleaning purposes. When the pipes became blocked, these plugs were found to be invaluable. The Canniff machine consisted of a steel tank, 2 ft. 7½ in. high and 18 in. in diameter, fitted with a movable hopper on top of a hinged door, a grating 9 in. from the bottom, to prevent lumps from going through the pressure pipes, and an air diffuser again below, which also served the purpose of a grating. The pressure was admitted to a T again by an ordinary coupling, and had a screw-down valve on each side and, in addition, a straight-way valve on the grout exit pipe. By a simple regulation of these valves, air pressures could be admitted either at the top or bottom, by which means the grout was prevented from settling and the clogging of the pipes was reduced to a minimum. Legs were provided on the bottom, and on the top flange rings for slinging, so that the machine could be moved readily by a crane. The tank was built in the Government workshops and tested at a pressure of 100 lb. per sq. in., the maximum working pressure being 75 lb. per sq. in.

Grouting Under Pressure.—A systematic plan of grouting was arranged (Fig. 3). By using fluorescein, a flow of water had been indicated in a cave subsequently found near the locality of the temporary pumps (first position); in addition to this, a flow of water was

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 404.

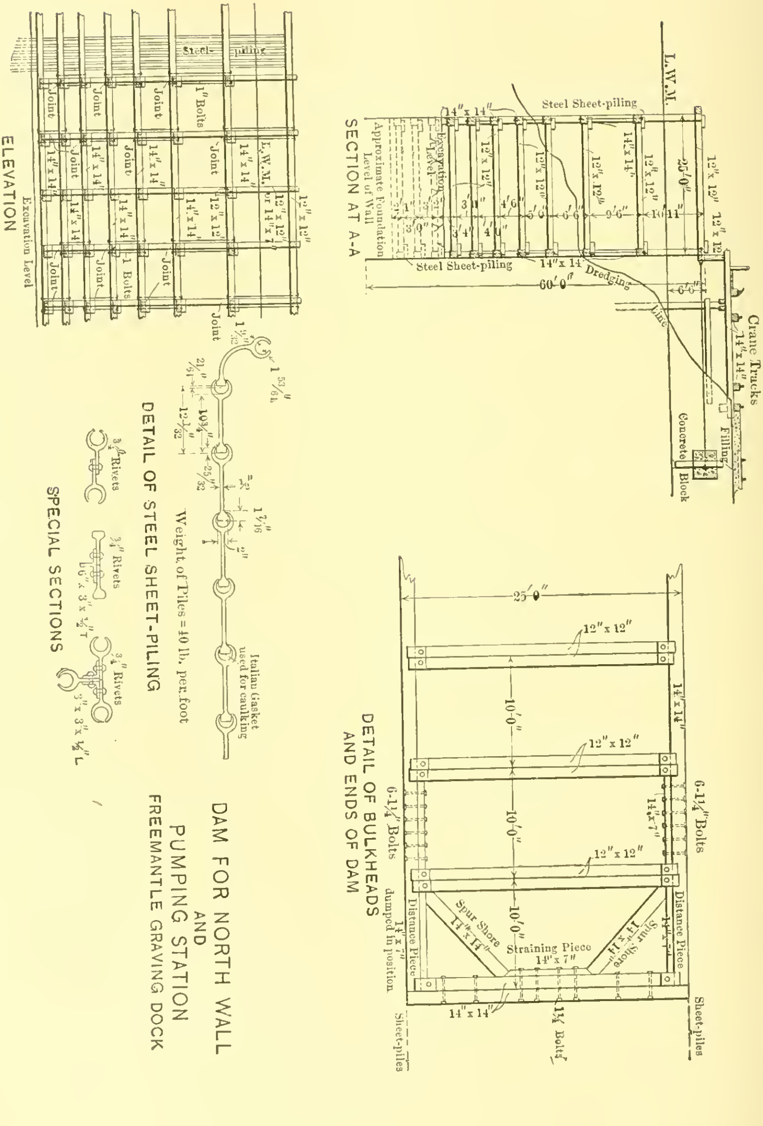
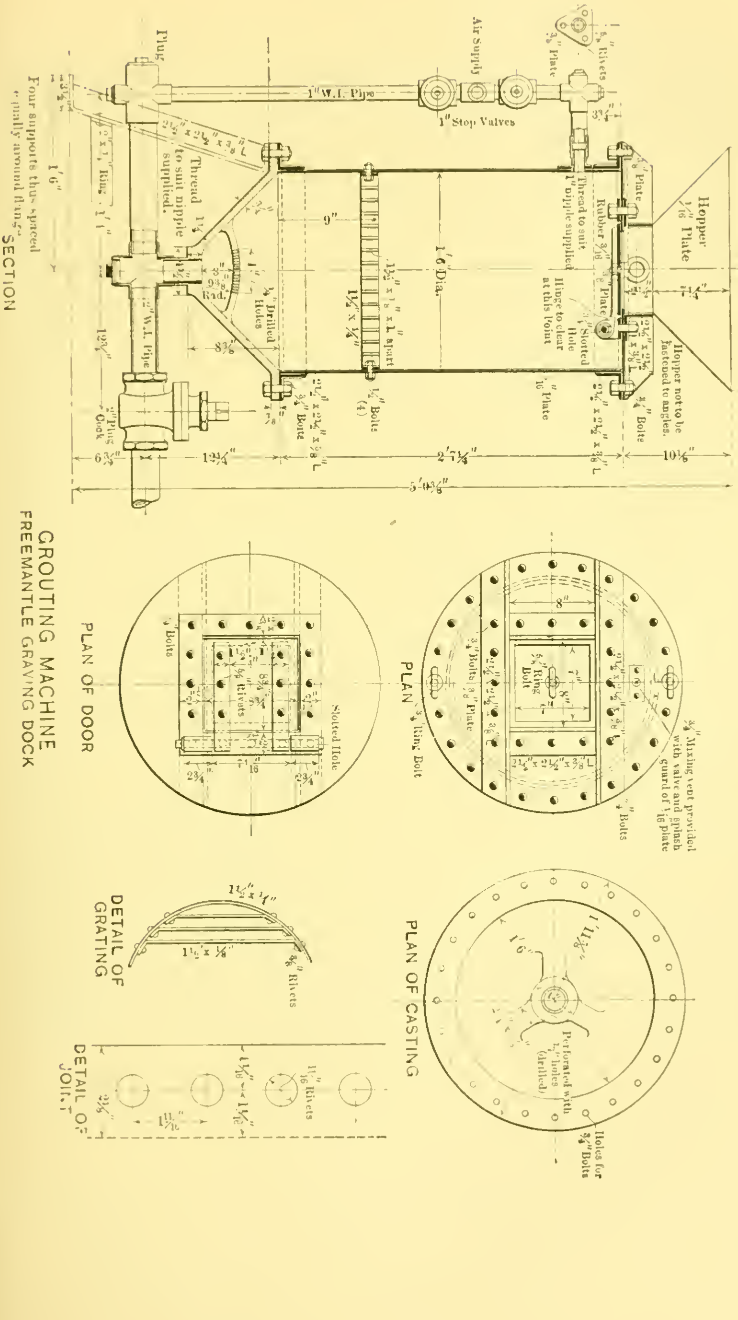

DAM FOR NORTH WALL
AND
PUMPING STATION
FREEMANTLE GRAVING DOCK
SECTION AT A-A
DETAIL OF BULKHEADS
AND ENDS OF DAM
ELEVATION
DETAIL OF STEEL SHEET-PILING
SPECIAL SECTIONS
L.W.M.
Steel Sheet-piling
Crane Tracks
Filling
Concrete Block
Dredging Line
Excavation Level
Approximate Foundation Level of Wall
Weight of Piles = 40 lb. per foot
Italian Gasket used for caulking
Spur Shore
Straining Piece
Distance Piece
Sheet-piles

FIG. 4.



GROUTING MACHINE
FREEMANTLE GRAVING DOCK

FIG. 5.

also found opposite the temporary pump-well in grouting the bore marked *C* on Fig. 3.

On grouting the west bores next to the sheeting, it was found that the flow of water had been entirely stopped. This was proved conclusively by the fact that when grouting holes were drilled 2 ft. 6 in. behind them, no water spouted up, and, further, the drilling was done through layers of hard cement; this was most encouraging.

As regards grouting the east side: A strong flow of water was observed, particularly in those grouting holes near the northeast corner, and the entire line was grouted up solid by the machine working from one hole in the center, showing that considerable penetration could be had for the cement.

As a precaution, all valves on the grouting pipes in the vicinity of the pipe being grouted were left open unless they were running freely. In the latter case one valve nearest the pipe being grouted was left partly open. The reason for this was that with a grouting pressure of 75 lb. per sq. in., there would be grave danger of blowing up the ground which is being grouted, and a valve in the vicinity left open would act as a safety valve. In actual practice it was observed that the surface ground started to crack, due to being in tension, and, but for turning off the pressure promptly, trouble would have been experienced, everything indicating the formation of a miniature volcano.

On an average, each hole took 43 cu. ft. of grout (equal parts of cement and water), and, when the machine ran very freely, saw-dust was added. From subsequent excavations it was shown that the results were most satisfactory, and the penetration of the cement most marked.

An appreciable decrease in pumping was now observable, but the flow in the well was still considerable. It was decided to put down some trial holes in the neighborhood of the burst, and Holes *A* and *B*, were drilled, as shown on Fig. 3. While drilling Hole *A*, the drill dropped 20 ft. 6 in. without any warning, indicating that a fissure or cave had been entered, the invert being 67 ft. 5 in. below low-water mark. Hole *B*, which was 4 ft. 9 in. from Hole *A*, indicated by a sudden drop of the drill that the cave entered was at that place 10 ft. deep, the invert being 57 ft. 5 in. below low-water mark.

A larger hole was then drilled into the cave and a 4-in. pipe was inserted. The water rose in this pipe to within a few feet of the

DRILLED HOLES SUBSEQUENTLY GROUTED SUCCESSFULLY.

level of the water in the river, and the pressure was found to be 9.53 lb. per sq. in.

In one of the grouting bores a pressure of 5 lb. per sq. in. had been recorded. It was realized that, unless this cave could be grouted, the chances of going on with the dock successfully were remote, and, further, that the utmost caution would be necessary in proceeding with the work. A sudden blow might very easily have meant a disaster to all the men working below. Careful preparations were made for filling the cave. A swab of Italian gasket with a sinker attached to marline was forced down the pipe into the cave, and there it was taken by the current and jammed into the exit or burst in the well. Altogether, sixty of these swabs were used, and in this way the burst into the well from the cave was sealed. Then the task of filling the cave was commenced. As previously mentioned, there were two pipes into the cave, on one of which a gravity pipe was fixed above ground level, consisting of a 1-yd. skip fitted with a pipe and valve in the bottom, and on the other the grouting machine was attached with the usual valve.

The fight was started in the early hours of the morning and was continued without stopping until midnight. Altogether, 29 tons of cement were forced into the cave; and, at midday, ashes and water were forced through the gravity machine with a view of saving cement, 6 tons of ashes being used.

A sounding was taken in the well at midnight, and no progress at all was observed. On sending a diver down to inspect the well, he reported that there was practically no leakage, so that some benefit had been attained indirectly.

At this stage several crucial points had been definitely established:

1.—The job would have to be dealt with in small sections.

2.—That being the case, with the pumping plant available, it would take at least 8 years to complete the work, and to overcome this it would be necessary to multiply the pumping plant in order to get several sections under way at a time.

3.—The cave would either have to be grouted up or opened out and filled with concrete. As the former had failed, the latter would have to be attempted, and would be extremely difficult.

4.—The ground was of such a doubtful character as to render it necessary to grout all over the site, which having been done, the safe

permissible load that it would carry would be low, and the writer was doubtful whether 3 tons per sq. ft. could be exceeded.

5.—Corrosion due to sulphureted hydrogen gas was going on so rapidly as to render the re-using of the front steel piles more than doubtful.

6.—Steps would have to be taken to decrease the pressure of the sulphureted hydrogen gas. This gas caused the writer considerable anxiety, and it is necessary to remember that the stability of the floor depended on the concrete being fortified by steel, and such fortified concrete had to be designed to withstand a hydrostatic pressure from below and a statical load due to a vessel on the keel blocks.

This latter was necessary owing to the ascertained uncertainty of the ground and the presence of caves. Therefore, if this steel in the ferro-concrete was attacked by corrosion accelerated by sulphureted hydrogen gas, the consequences would be serious.

A supplementary estimate was given to the Government, which brought the estimated cost of the dock from its inception up to £725 000, and the writer advised the Engineer-in-Chief to recommend the Government to abandon the site and look elsewhere.

From a harbor point of view, the site was a bad one, and if, as the writer firmly believes will happen, liners in the Australian trade materially increase in size, it is more than probable that either the dock or the Port of Fremantle would have to be moved.

Bearing in mind that a great deal of the work done on the site, such as dredging, would be reproductive, the Government decided:

1.—To look elsewhere for a dock site;

2.—To build a quay on the north side of the dock site capable of berthing any mammoth liner afloat, and as such berth was at the mouth of the river, it would entail the minimum amount of dredging for an entrance to the deep-water berth; and

3.—To dredge away everything to the south and west and so very materially improve the harbor.

It is never satisfactory to abandon any engineering work; nevertheless, it is the duty of the engineer to keep a careful watch, and forecast the ultimate expenditure of the work.

Taking all items into consideration, everything indicated that the ultimate cost of the dock would be exceptionally heavy, and, at the

same time, the probable return on the money expended through steamers utilizing the dock would be small; consequently, the Government was fortunate in being able to take advantage of the alternative scheme and utilize the work already done.

The latent resources of Western Australia, such as timber, mining, agriculture, dairying, wool, and fruit, are being rapidly developed by a progressive Government, and a further extension of the Harbor of Fremantle has been authorized.

Table 1, is a schedule of the rates of pay for the various classes of workmen engaged on the work, and may be of interest. The hours worked totaled 48 per week, and, in addition, there were 10½ paid holidays per annum.

TABLE 1.—Rates of Pay of Workmen.

Class.	Rate per day, in shillings.	Class.	Rate per day, in shillings.
Shift foreman	20	Engine drivers	12
Foreman laborer	14	Crane drivers	12
Sub foreman laborer	11	Loco. drivers	12
Foreman carpenter	14	Loco. firemen	10
Foreman shipwright	14	Sawyer	11
Foreman fitter	15	Sawyer's laborer	9
Leading hand fitter	13	Wire splicer	10
Yard watchman	8	Sailormen	9½
Chainmen	9½ and 10	Plate-layers	9½
Storekeeper	12	Dam watchmen	9½
Storeman	10	Divers	20
Timekeepers	11 and 12	" tender	10
Clerks	10 and 11	" assistants	9½
Carpenters	12	" pumpers	9½
Blacksmiths	12½	Plumber	13
" striker	9½	" laborer	9½
Fitters	12	Timbermen	9½
" laborers	9	Foremen pile-drivers	11½
Firemen	10	Pile-drivers	9½
Trimmers	10	Laborers	9
Crane attendants	9½	Drillers	9

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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NOTES ON BRIDGEWORK.
Discussion.*

By S. Vilar y Boy, Assoc. M. Am. Soc. C. E.†

S. Vilar y Boy, Assoc. M. Am. Soc. C. E. (by letter).—Regarding the four conditions required to deal with a continuous beam as though it were two independent beams fixed at the intermediate support, Conditions 1 and 3 are of course fulfilled and also Condition 2, because, when settlement is feared, a structure is not supported at different levels, but on the same level; and it is under the condition of the same level that loads act on the structure, which will settle unevenly if the foundations are not well proportioned to the loads they are to sustain. This is what must be avoided; therefore, it cannot be considered as an actual working condition. Mr. Vilar y Boy.

As for Condition 4, it is not mathematically fulfilled, but might be accepted as approximate enough, because the position of the engine for the maximum reaction on the intermediate support agrees very nearly with it, for which the writer refers to Fig. 2 and to Article 15 of "Railroad Construction," by W. L. Webb, M. Am. Soc. C. E., in regard to the possibility of assuming uniform axle loads, uniformly spaced, in computing trestle works, with results approximate enough in actual practice, which is the writer's standpoint, as stated in the preface of the paper.

Moreover, it is usual, in working out solutions of engineering problems, that some assumptions be made, for the sake of simplicity in the resulting formulas, provided they prove to be approximate enough in practice.

In quoting the equation for the reaction at A, Mr. Howalt shows $\frac{W b}{a + b} - \frac{Z}{2}$ instead of $\frac{W b}{a + b} - Z$, according to the writer's development; that is the point in discussion, and the uplift, $-Z$, has been taken into account when calculating the reaction.

* Continued from October, 1912, *Proceedings*.

† Author's closure.

Mr. Vilar y Boy. As for Fig. 3, which was used as a loading diagram to compute the cross-section of beams, the writer accepted it in spite of the fact that it is not absolutely exact, either for the bending moment or for the end reaction, because, if refined accuracy is not required, it is rather usual to compute the bending moments by placing the heaviest axle loads about in the center of the span, and the others in their relative positions. As for the reactions, they are calculated for the heaviest total load on the beam, and, after stating that the formula obtained for the moment, M, ought to be disregarded in practice, the writer did not consider it necessary to go fully into the matter, as everybody is acquainted with the influence diagrams and the method of the French engineer, M. Barré, of fixing the position of the center of gravity of the axle loads for the maximum bending moment on end-supported beams.

The foregoing assumption might be checked as follows:

Let $M = 38.7$ tons, as accepted in the writer's calculations, so that $M = 19.3$ tons under each rail $= 232$ in-tons $= 0.5$ tons (permissible stress) $\times \frac{I}{Z}$ (section modulus). Therefore, $\frac{I}{Z} = 464$, which practically corresponds to two 6 by 16-in. pieces, the section modulus of which is 512.

Using the bending moment, $M = 42.07$ tons, as worked out by Mr. Howalt, M, under each rail $= 21.03$ ft-tons $= 252.36$ in-tons, and assuming as before that $S = 0.5$ tons, $\frac{I}{Z} = 504.72$, which also corresponds to two 6 by 16-in. pieces.

Therefore, the writer's purpose being only the comparison of the resulting cross-sections for the same loading diagram, both in the case of the end-supported beam and of the one fixed-end beam, the use of Fig. 3 does not appear to be unreasonable.

The value of $M = 30$ ft-tons does not apply to Fig. 2, as Mr. Howalt appears to think, but to Fig. 3, which might be ascertained by checking up the arithmetical calculations, so that really it cannot be compared to the value, $M = 49.92$ ft-tons, obtained by Mr. Howalt for Fig. 2, even if such value be correct.

The writer does not like to insist on trifles, but his attention was called to the fact that the bending moment for a continuous beam under concentrated loads is higher than for an end-supported beam covering the same span at the rate of 49.92 ft-tons, as against 42.07 ft-tons (both figures calculated by Mr. Howalt), and although the matter has been explained quite definitely by Mansfield Merriman, M. Am. Soc. C. E.,* the writer checked up the foregoing figures as in Fig. 5.

* "Mechanics of Materials," Chapter on "Comparison of Beams."

Mr. Vilar y Boy.

The central load of 12 tons being disregarded, as it bears directly on B, and, therefore, produces no deflection, the theorem of three moments for concentrated loads might be applied to the two 11-ton axles, and therefore:

$$M' l' + 2 M'' (l' + l'') + M''' l'' = - P' l'^2 (K - K^3) - P'' l''^2 (2 K - 3 K^2 + K^3).$$

As the beam is simply supported at the ends, $M' = M''' = 0$. Also, $l' = l'' = 12$ ft., $K = 0.54$ for the first span, and $K = 0.5$ for the second span.

Then:

$$48 M'' = - 11 \times 144 (0.54 - 0.16) - 11 \times 144 (1 - 0.75 + 0.125)$$
$$48 M'' = - 11 \times 144 (0.54 - 0.16 + 1 - 0.75 + 0.125)$$
$$= - 11 \times 144 \times 0.755 = - 1\,195.92 \text{; and } M'' = - \frac{1\,195.92}{48}$$
$$= - 24.91 \text{ ft-tons, as against } 49.92 \text{ given by Mr. Howalt.}$$

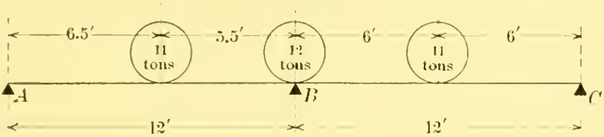

FIG. 5.

Dealing with the continuous beam as though it were two separate beams fixed at the intermediate support, and applying the writer's formula for M, the results are as follows:

For the first span = 25.2 ft-tons;
For the second span = 24.7 ft-tons;

as against the 24.91 ft-tons given by the theorem of three moments, whereas Mr. Howalt's figures are about twice as great.

As the obtained value for the reaction at B "happens to be correct," and the value of M, according to the same method, also happens to be very close to the exact figures, and, moreover, as the whole deduction is in accordance with the general principles of the theory of flexure, the writer thinks that his method is not altogether wrong, although he considers that in practice it must be disregarded in computing the cross-sections of stringers, as stated in the paper.

Referring to Mr. Parker's discussion, the only point to be decided is whether or not the results obtained by considering the continuous beam as two independent beams, fixed at the middle point, are approximate enough in practice. Of course, the deduction on the basis of the theorem of three moments is mathematically exact and excludes any assumption, and, furthermore, Mr. Parker's formulas are so handy that had the writer known of them, he would have accepted and used

Mr. Vilar y Boy. them instead of working out the solution of the same problem in a different way.

At the same time, Mr. Parker's conclusion as to the method in the paper giving a much less reaction at R_2 than the correct one, is probably due to the fact that the percentage shown by the curves in Fig. 4 refers to the actual load on the beam, whereas the percentage shown in the paper refers to the calculated reaction without regard to the continuity of the beam.

The figures obtained by applying Mr. Parker's formulas to Fig. 2 will prove it:

$R_2 = wa + \frac{1}{2} w (a - a^3)$ gives the following results:

For the left-hand load,	$R_2 =$	8.03 as against	8.06	shown in the paper.
For the right-hand "	$R'_2 =$	7.59 " "	7.56	" " " "
Load resting on B,	$R''_2 =$	12.00 =	12.00	" " " "
Total reaction at B,		27.62 =	27.62	shown in the paper.

In order to prove that the foregoing result is not a mere coincidence, the writer tried to compare both formulas (Mr. Parker's and his own) from a theoretical and abstract standpoint, as follows:

To avoid any misunderstanding, the symbols, a and b, in the writer's formula will be expressed in Greek letters, α and β, because in such formula a and b represent the actual dimensions, in feet, whereas in Mr. Parker's they are fractional factors, so that the actual lengths of the two stretches of beam are al and bl, respectively, that is to say, $\alpha = al$ and $\beta = bl$, and as $\alpha + \beta$ must be equal to the total span, l, it follows that $a + b = 1$.

Therefore, the writer's formula:

$$R_2 = \frac{W\alpha}{l} + \frac{W\alpha\beta\,(l + \alpha)}{2\,l^3}, \text{ will be easily reduced to:}$$

$$R_2 = Wa + \frac{W \times al \times bl\,(l + al)}{2\,l^3} = Wa + \frac{W l^3 \times ab\,(1 + a)}{2\,l^3}, \text{ or,}$$

$$R_2 = Wa + \frac{W}{2} \times ab\,(1 + a), \text{ but } b = 1 - a, \text{ so that}$$

$$R_2 = Wa + \frac{W}{2} \times a\,(1 - a)\,(1 + a) = Wa + \frac{W}{2} \times a\,(1 - a^2)$$

$$R_2 = Wa + \frac{W}{2}\,(a - a^3)$$

which is identical with the resulting formula on the basis of the theorem of three moments, this being why the writer believes that Mr. Parker's assertion, about the different results obtained from his and the writer's formula, is due to a misunderstanding.

Similarly, the writer's formula for R_1 might be reduced to: Mr. Vilar y Boy.

$$R_1 = w\,b - \frac{1}{2}\,w\,(a - a^3) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

against $R_1 = wb - \frac{1}{4}\,w\,(a - a^3)$ from the theorem of three moments, the difference between these two values being $-\frac{1}{4}\,w\,(a - a^3)$, the maximum of the absolute value of which (for a constant load, w) corresponds to the maximum value of $(a - a^3)$, the first derivative of which:

$1 - 3\,a^2$ equalized to 0, that is to say, $1 - 3\,a^2 = 0$, solved for a leads to $a = \sqrt{\frac{1}{3}} = \sqrt{0.3333} = 0.58$, which corresponds to a maximum of the function, as the second derivative is a negative expression.

Therefore, the maximum absolute value of $\frac{1}{4}\,w\,(a - a^3)$ should be: $\frac{1}{4}\,w\,(0.58 - 0.20) = \frac{1}{4}\,w \times 0.38 = 0.095\,w$, or, in other words, the maximum error in the value of R_1, due to the writer's assumption, should be 9½% of the load, w, in case only one span is loaded, as shown in Fig. 4.

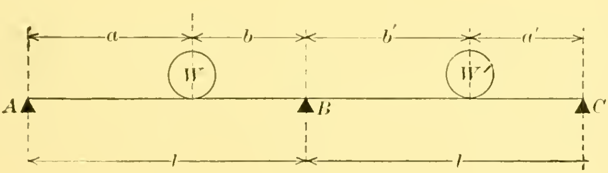

FIG. 6.

If the two spans are loaded, as in Fig. 6, the reaction at A should be as follows:

On account of the load, W, $r_1 = W\,b - \frac{1}{4}\,W\,(a - a^3)$

.. .. W', $r_1' = (\quad) - \frac{1}{4}\,W'\,(a' - a'^3)$

and, therefore, the total reaction at A would be:

$$R_1 = W\,b - \frac{1}{4}\,W\,(a - a^3) - \frac{1}{4}\,W'\,(a' - a'^3)$$

and, if $W = W'$, $a = a'$, $b = b'$, or, in other words, if the spans be symmetrically loaded: $R_1 = w\,b - \frac{1}{2}\,w\,(a - a^3)$, which is identical with Equation (4), so that if the axle loads are uniform and uniformly

Mr. Vilar y Boy.

spaced, which is the case of the engine drivers in the standard loading diagram, both formulas (Mr. Parker's and the writer's) should lead to exactly the same values.

As for the actual axle loads, the conditions prove to be so close to symmetry that the difference between the results of both formulas may be disregarded in practice.

Referring again to Fig. 2:

$a = 0.54$, $b = 0.46$, $a' = 0.5$, $b' = 0.5$, $W = W' = 11$ tons, so that by using Mr. Parker's method:

$$R_1 = 11 \times 0.46 - \frac{1}{4}\, 11\, (0.54 - 0.16) - \frac{1}{4}\, 11\, (0.5 - 0.125)$$

$$= 11\,(0.46 - 0.095 - 0.094) = 11 \times 0.271 = 2.981 \text{ tons.}$$

Whereas, the writer's method gives:

$$R_1 = \frac{11 \times 5.5}{12} - \frac{11 \times 6.5 \times 5.5 \times 18.5}{3\,456} = 11 \times 0.46 - 11 \times 0.191$$

$$= 11 \times 0.269 = 2.959 \text{ tons.}$$

The error is $2.981 - 2.959 = 0.022$ tons, that is to say, less than 1% of the exact value, which is within the error of the slide-rule, the use of which is generally accepted.

Reviewing the whole discussion, the following conclusions might be obtained:

First.—As for the value of the reaction at the intermediate support (which was the writer's primary purpose), the assumption of a continuous beam being equivalent to two separate beams fixed at the intermediate support proves to be absolutely exact in any case.

Second.—As for the reaction at the ends and the bending moment, the above assumption is absolutely correct in the case of the two spans being symmetrically loaded, and approximate enough in practice for the actual loading diagrams.

Third.—As for the reaction at the ends and the bending moments, the assumption should be incorrect in the case of only one span being loaded, which is not the actual loading diagram in the problem submitted by the writer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE STRENGTH OF COLUMNS.

Discussion.*

By W. E. Lilly, Esq.†

W. E. Lilly, Esq. (by letter).—Mr. Godfrey's discussion contains a great deal of controversial matter regarding the strength of columns, and the writer will endeavor to reply to some of the points that have been raised. It is to be regretted, however, that Mr. Godfrey was not more definite in stating his objections. Vague statements, unless based on reasoning of some kind or experimental evidence, can hardly be considered as discussion on the subject. Mr. Godfrey states that he has repeatedly assailed both the Rankine-Gordon and Euler formulas for columns, and has observed and demonstrated, in his practice and reading, that confidence in these formulas has resulted in failure. The writer totally differs with him, after an examination of the majority of the tests made on columns. From the results of his own experiments, he has come to the conclusion that the Rankine-Gordon formula, when secondary flexure is allowed for, may be relied on for the design of columns; and, as he has shown in previous papers, it gives results which agree more closely with the experimental tests than can be obtained by the use of any other formula; further, the formula is logically derived, a merit not possessed by any other column formula. He has fully discussed the derivation of both the Rankine-Gordon and Euler formulas in the papers referred to in the Appendix. Mr. Lilly.

Mr. Godfrey's examples of the uses of Rankine-Gordon's and Euler's formulas are unfortunate, and show how dangerous it is to use the constant in the Rankine-Gordon formula without knowing how it has been derived. Euler's formula always gives a greater value than that obtained from the Rankine-Gordon formula under similar conditions of column loading. The writer would refer Mr. Godfrey

* Continued from December, 1912, *Proceedings*.

† Author's closure.

Mr. Lilly. to the constants given in the paper and to the manner of their derivation.

Mr. Godfrey states that the Rankine-Gordon formula fails to meet the needs of the practical design of columns, and that the straight-line formula is far superior. It gives results closer to those obtained from experiments, and there are several reasons why it should. In the paper the writer has referred to the merits of the Rankine-Gordon formula as a practical one for the design of columns, and does not propose to discuss it further. He has also referred to the straight-line formula. His own experiments show that it does not give results which agree closely with the experiments, and there are no reasons, based either on theory or experimental evidence, why it should.

Mr. Godfrey mentions two instances of columns, in which the component parts were joined by batten-plates and of the disastrous results which followed the use of these columns. The writer agrees with Mr. Godfrey that lattice bracing should be used, as columns of this kind cannot transmit the shear stresses set up by the bending of the columns.

Mr. Eckersley's remarks are generally in accordance with the writer's views; the mathematical treatment of the column problem, however, in the present state of knowledge, appears to be so full of difficulties as to be almost incapable of rigorous solution, notwithstanding the efforts made in this direction by various investigators. It is for this reason that so many semi-empirical formulas have been put forward at different times, and of these, as stated in the paper, the writer considers the Rankine-Gordon formula, together with his own modification of it, to be the best.

He thinks that any attempt to solve the column problem should be approached somewhat as follows: Let a solid column, as shown by Fig. 1, be given, in which the load is applied at a given eccentricity, e. When loading is gradually applied to the column, it commences to bend or deflect, and the magnitude of the deflection depends on the elasticity of the material, other quantities being assumed as constant. As the loading is increased, the deflection increases, and when it reaches a limiting value, depending on the strength of the material, failure of the column takes place. The problem thus considered shows that, among other quantities, three are involved, namely, the coefficient of elasticity, E, the strength of the material, f, and the deflection, d, when failure of the column takes place.

It follows that any column formula must of necessity, among other quantities, include these three, if it is to be generally applicable. The mathematical treatment of the problem leads to difficulties in determining the deflection, d, and as far as the writer is aware, it can only be approximated to. It remains for future investigators to determine the rigorous form of the function governing the deflection.

The formation of a satisfactory column formula will then be an easy matter. Mr. Lilly.

Mr. Prichard's remarks require a more detailed reply, and the writer will endeavor to discuss the more salient points raised. Mr. Prichard states that he "does not share the author's opinions as to the theoretical and practical excellence of the Rankine formula, and he does not approve of the use of such a high unit stress for the 'strength to compression' "; and, further, after referring to Rankine, he states:

"Subsequently, he [Rankine] makes the mistake of applying one of these equations to the deflection of columns, entirely overlooking or neglecting the fact that, in all cases of ordinary columns, part, at least, of the stress in the extreme fiber is due to direct compression. This mistake leads to an erroneous formula for columns, which, on the strength of Rankine's endorsement, has deceived and has continued to deceive engineers, and interfere with a correct understanding of the subject, for more than fifty years.

"The error is reflected in the author's Equation 6 for deflection of columns, which he makes a factor of the total stress intensity in the extreme fiber, f, instead of a factor of $f - p$, as it should be (p being the load per unit area). The author prefaces this substitution with an unwarranted assumption, which, as he states, 'errs', and gives what he terms 'the first approximation'."

With regard to the foregoing remarks, either Mr. Prichard is in error, or he has not studied the underlying assumptions made in the derivation of the Rankine-Gordon formula; for the real point of the formula is that it does allow for the stress in the extreme fiber due to direct compression. Mr. Prichard is also in error in stating that $f - p$ should be used for p in the derivation of the formula, and the writer would refer him to the articles in *Engineering,* quoted in the appendix to the paper, in which he will find the Rankine-Gordon and Euler's formulas fully discussed. There is no doubt that, after perusing the same, he will qualify his opinion regarding the substitution referred to being an unwarranted one. Mr. Prichard then quotes from Rankine the theorem underlying the Eulerian analysis of a spring or slender column, and states:

"In view of the prevalent uncertainty as to the theory of perfectly elastic, initially straight, centrally loaded columns, the writer suggests that the author state whether he accepts this theorem of Rankine's; and if he does not accept it, that he indicate wherein he considers Rankine's 'proof' to be in error."

In the articles in *Engineering* previously referred to, the writer has already stated his objections to the Eulerian analysis, and pointed out wherein it errs; the assumption of I, the moment of inertia, remaining constant when applied to columns is false; also the value of E does not remain constant outside the range of the elastic limits. An article

Mr. Lilly. in *Engineering** on the strength of columns will give Mr. Prichard further information on this point.

Mr. Prichard does not agree with the writer's high unit stresses adopted for the strength to compression, and states:

"The yield points of wrought iron and steel are, beyond question, the critical stresses for wrought-iron and steel columns, and should be substituted for the crushing resistances given in Table 1 as 'the strength to compression'."

The writer does not agree with this statement; from his own experiments he is of the opinion that the yield point can only be determined with any degree of accuracy on annealed bars. On bars which had been subjected to cold rolling or to permanent strain he doubts whether any real yield point could be said to exist. He thinks that too much importance has been attached to this quantity as a standard measurement. Tables 2 and 3, given by Mr. Prichard, show a variation of the values from 29 000 to 49 000 lb. per sq. in., and, in view of such wide variations, the writer prefers to retain the values for the strength to compression given in the paper. He would refer Mr. Prichard to a paper on the elastic limit and strength of materials,† in which the above points are more fully discussed.

Mr. Prichard refers to the formula for columns given by Marston, and states, in Table 5, that this formula was first given by Marston in the *Transactions*, Am. Soc. C. E., Vol. XXXIX, and since endorsed by many authorities. Mr. Prichard is in error; the formula was given, as long ago as 1878, by Smith,‡ and modifications of it have been given by Perry and Neville. Mr. Prichard puts forward this formula as a rigorously theoretical one; this, however, is not the case, as the writer showed in the articles in *Engineering* already referred to. Considered from the point of view of a practical column formula, it is open to many objections. The values given in Table 5 show this. In the particular case when the eccentricity, $e = 0$, the loads on the column, for values of $\frac{l}{\rho}$ between 0 and 70, are all equal, and, for values greater than 70, the loads follow the Eulerian curve; this is equivalent to assuming that, for values of $\frac{l}{\rho}$ between 0 and 70, the column has no deflection when failure takes place, and then for values of $\frac{l}{\rho}$ greater than 70 for the deflection to vary approximately as the square of the length of the column. This is contrary to experience, as shown by tests on columns, and is evidently incorrect. To assume some value

* August 23d, 1912.

† Read before the Institution of Civil Engineers of Ireland, December 6th, 1911.

‡ *Proceedings*, Edinburgh and Leith Engineering Society.

for the yield point of the material from experiment, and then to assert that no column within the range of $\frac{l}{\rho}$ between 0 and 70, when loaded with the load per unit of area, will bend, is inconceivable; and yet this is the standpoint from which Mr. Prichard bases his faith in the Smith-Marston formula. Experiments show that columns always bend before failure takes place, and the writer is of the opinion that some function of the length must be assumed for the deflection in the derivation of any useful column formula. It is in this respect that the Eulerian analysis has had such a retarding influence on investigations on columns, in the enunciation of a critical load as obtained from Euler's formula, together with a zero deflection of the column, in contradistinction to the idea that the essential point to be considered is the deflection of the column when failure takes place. The Smith-Marston formula has never found favor with engineers, owing to its being troublesome to use in practice; if it were theoretically correct, this would be no argument against it, but, as it is not, there is nothing to be gained in using it. In conclusion, Mr. Prichard states: Mr. Lilly.

"An important step in this direction will have been taken when it becomes generally recognized that Rankine's formula is based on a blunder caused by a plausible but fallacious assumption of analogy in deflection of beams and columns, that it has only the semblance of a 'theoretic basis', and that it is not even a good empirical formula for wrought iron and steel."

The statement of the derivation of the Rankine-Gordon formula is incorrect; if anything, it applies more nearly to the formula advocated by Mr. Prichard, as the writer has shown in the articles already referred to.

Mr. Branne's remarks are of interest to the writer. As far as he is aware, the Rankine-Gordon formula is the only one which permits the determination of the sizes of the bracing in a logical way, and the examples given in the paper show that the results obtained by its use are closely correct. From a comparison of the tests carried out on columns, it is difficult to generalize, owing principally to the limited range of the length of the columns tested, and it is to be hoped that, in the future, in arranging for further experiments, this point will be borne in mind.

In conclusion, the writer thanks those gentlemen who have discussed the paper. Some difference of opinion was to be expected, the subject being one of much complexity. If the paper leads to a co-ordination of the results of the tests on columns, the writer will feel that he has been amply repaid for the labor of preparing it.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

A BRIEF DESCRIPTION OF A MODERN STREET RAILWAY TRACK CONSTRUCTION.

Discussion.*

By C. B. Vorce, M. Am. Soc. C. E.

C. B. Vorce, M. Am. Soc. C. E. (by letter).—The writer has read Mr. Polk's paper with much interest, and is glad that a matter of such vital importance to so many members has been brought to the attention of the Society. Mr. Vorce.

As an expenditure of many thousands of dollars is involved, a very careful study should be made before a type of permanent track construction is adopted. In many cases, failure has occurred by reason of a lack of study of the character of the foundations, and a type not at all suitable to existing conditions has been chosen simply because other railroads have used that type successfully.

The writer has had very poor results with the girder type of construction adopted by Mr. Polk. This type was the standard when the writer became connected with the British Columbia Electric Railway Company, Limited, of Vancouver, but, on account of poor foundations, the girders had broken badly, with most disastrous results to both pavement and track.

After a careful study, the writer recommended, and the management adopted, the type of construction shown on Fig. 4. Since that time about 15 miles of double track of this type have been laid, and as yet no signs of failure can be seen.

It is the writer's opinion that it is possible to build too rigid a track. He is trying to avoid a hard, rigid, inelastic track, which increases both car and track maintenance, and for this reason he criticizes the steel ties embedded in concrete used by Mr. Polk. The concrete

* Continued from December, 1912, *Proceedings*.

Mr. Vorce. between the ties and under the rail will shrink, allowing the rail to move slightly. The writer is in favor of wooden ties, spaced 2 ft. from center to center, surfaced on about 1 in. of sand, which acts as a cushion. This type has given excellent satisfaction in Vancouver, and is not as noisy and rigid as the steel tie construction.

As to the life of the wooden ties: On special work in Vancouver, ties which have been embedded in concrete for 14 years have been taken up and found to be perfectly sound.

Although Mr. Polk goes into seemingly great refinement to get a tight joint, he does not mention the grinding of the rail ends so that each shall be in the same horizontal plane, which is a matter of vital importance. The rail ends vary, and, no matter how slight this variation may be, unless they are ground to the same height, pounding at the joints will occur, cupping will commence, and the track will fail at the joints while the remainder is in perfectly good condition. When it is remembered that the joints are only about 6% of the total track length, it is readily seen to be good business to make the joints as nearly perfect as possible.

Although the writer's company has bought the best paving bricks obtainable on the Pacific Coast, the results with them in forming a flangeway on the running side of the rail have been very poor, as the bricks break off at the nose very quickly; for this reason the company has been forced to adopt granite flangeway blocks, although the cost is at least 50% greater.

As many of the streets in Vancouver are paved with wood blocks, which are narrower than the granite flangeway blocks, no bond is possible, and a straight joint parallel to the rail results (Fig. 1, Plate XXXVIII). With the girder construction, it is almost impossible to keep these flangeway blocks in place, due to the moving of the rail on account of broken girders. Since the adoption of the slab type of construction, which gives a solid foundation, there has been very little trouble.

During the coming season the writer proposes to lay a concrete monolith, 8 in. wide, parallel to the running side of the rail, in which the flangeway is formed. As this will be laid at the same time as the pavement foundation and form a part of it, it is hoped that this will overcome the difficulty of loose flangeway blocks, and reduce the maintenance charges.

In selecting a type of permanent track construction in Vancouver, one must remember that the conditions are not the same as in older established cities where macadamized streets have been in existence for years and a good foundation is available. Owing to the poor foundation, the streets were first planked and then macadamized, the macadam seldom being renewed before laying the permanent pavement on a concrete base. For this reason the foundation is very poor, and the

FIG. 1.—LAYING GRANITE FLANGEWAY BLOCKS ON GRANVILLE ST., VANCOUVER, B. C.

FIG. 2.—TRACKS REMOVED—GRADING IN PROGRESS—TEMPORARY TRACK ON LEFT.

FIG. 3.—GRADING COMPLETED. TEMPORARY TRACK ON RIGHT, BROADWAY, VANCOUVER, B. C.

Mr. Vorce.

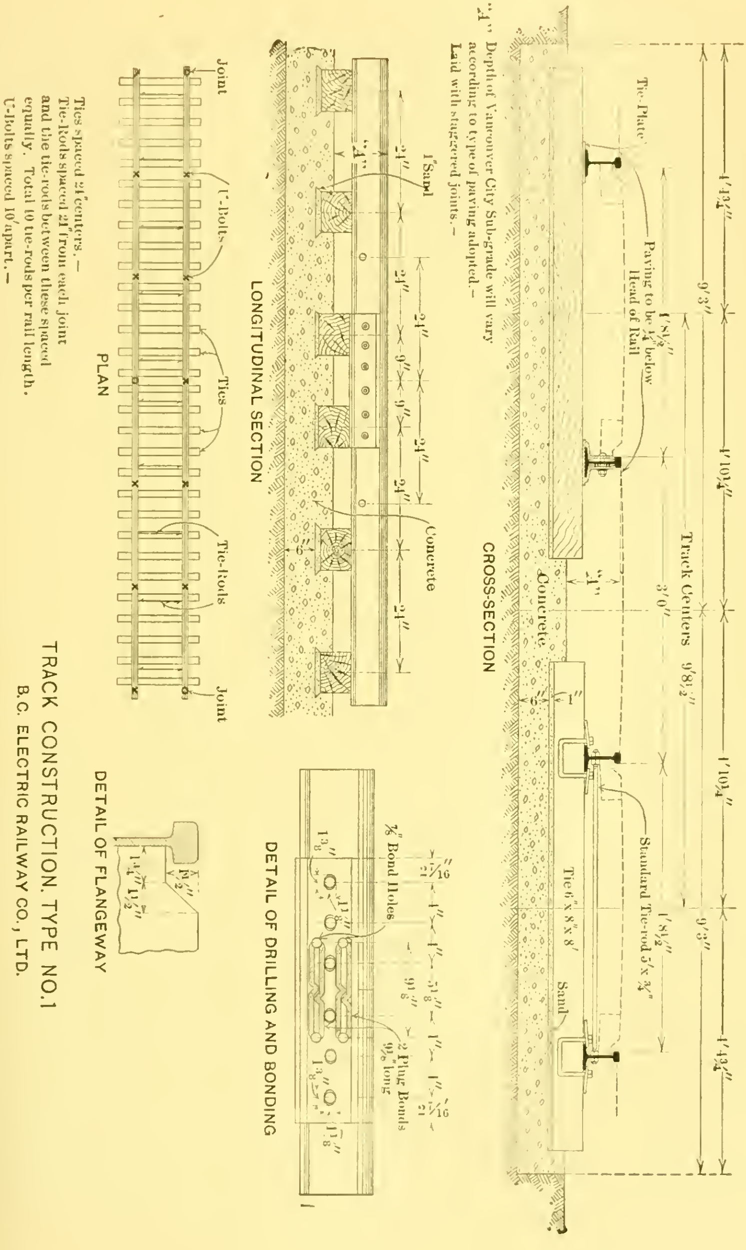

FIG. 4

Mr. Vorce. problem has been the selection of a type of construction to meet the existing conditions.

On paved streets, the company always lays a double track, and, as the City pays for all paving for the entire width between the curbs, including foundation, the railway company's liability covers only whatever construction it decides to put in below it. Before the writer took charge of this work, although under the franchise the company could do its own portion of the work, it had been the practice to let it to the paving contractor. Such poor results were obtained that the writer organized and equipped a construction department, and, with few exceptions, the company now does all its own work. At the suggestion of the writer, in order to facilitate the work, the company entered into an arrangement with the City to do all the work in the track allowance, about 18½ ft. wide, the City paying for its share at the same rate as was paid to the contractor for the remainder of the street. This enabled the railway company to do its portion of the work first, the contractor following up and bringing his work to the grade established by the rails. This plan has given most satisfactory results.

The work is carried on in the following manner: A temporary track is laid, outside of the track allowance, to take care of the traffic during construction, thus enabling the company to build both tracks at once. When the traffic is turned over to the temporary tracks, the old track is torn up and the ties and rails are piled on the side of the street, ready to be picked up by the work train, grading is then started (Fig. 2, Plate XXXVIII), and the cut is taken out to rough grade; forms are then set by the company's engineer to the proper grade for the top of the slab, and a small gang of fine graders follows up the rough graders; in the meantime the gravel for the slab is delivered along the side of the excavation (Fig. 3, Plate XXXVIII). As soon as a block has been rough-graded, work on the concrete slab is commenced, the material being shoveled directly into the hopper, on which a line has been drawn to insure the right proportions. The aggregate is then run through the mixer and delivered into a bucket which travels along a 30-ft. boom cable swinging through 180° (Fig. 1, Plate XXXIX). This method reduces the number of men around the mixer to the minimum, which is an important item, as common labor is paid 37½ cents per hour. All the company's mixers are of this type, and can mix about 80 cu. yd. per 10 hours, or about 250 lin. ft. of completed slab 18½ ft. wide and 6 in. deep. The rails are distributed by an electric derrick car and the ties by the company's motor car. As soon as the slab has set for 24 hours, ties are distributed, T-rails (70-lb., 264, Lorain Section, or 87-lb., 399, Lorain Section) are laid, and the track is surfaced to grade with sand (Fig. 2, Plate XXXIX). The standard plan calls for 1 in. of sand under the tie, but, owing to

FIG. 1.—LAYING CONCRETE SLAB, BROADWAY, VANCOUVER, B. C.

FIG. 2.—LAYING AND SURFACING PERMANENT TRACKS, BROADWAY, VANCOUVER, B. C.

irregularities in the concrete, the thickness varies from ¾ to 1¼ in. After the track has been surfaced, the sand is scraped out from between the ties, the flat wheels on the mixer are replaced by flanged wheels, so that the mixer can run on the track, and the final concrete course is laid (Fig. 1, Plate XL). Mr. Vorce.

It will be seen that the four operations, grading, laying the slab, laying the track, and putting in the final concrete course, are all carried on at the same time, resulting not only in speed but in economy.

Each section of track laid represents a new problem, and from time to time the writer has had to design special machinery in order to do the work quickly and economically. On Granville Street, the principal business street, though the paving was in good condition, about ½ mile of track was in such bad repair that it had to be renewed. The Board of Public Works allowed the company to abandon traffic on this street on condition that it would be restored in 22 days. As this period included 10 days for the last concrete laid to set, it meant that all work, except paving, had to be completed in 12 days. In order to be sure of carrying out the work on time, a progress chart, Fig. 5, was prepared, and was followed so closely that the work was finished 1 hour ahead of time.

The street was paved with wood blocks laid on a 6-in. concrete base, and each rail of the double track was carried on a concrete girder, 24 in. wide and 15 in. deep. The reconstruction of the track called for the removal and replacing of the following quantities: 5 100 sq. yd. of wood block paving; 10 200 ft. of rail; 1 800 cu. yd., or 5 100 sq. yd., of concrete; 600 cu. yd. of earth; the laying of 2 400 cu. yd. of concrete; ½ mile of double track; 850 sq. yd. of concrete flangeway blocks; and 4 250 sq. yd. of wood block paving. The quantity of concrete to be removed was so great, and the time allowed so short, that some mechanical means for breaking it up had to be devised, as the schedule called for the removal of about 32 sq. yd. per hour. The dipper arm was taken off the electrically-driven shovel owned by the company, and pile-driver leads, 20 ft. long, were hung from the end of the boom, being held in place by two braces running back to the main frame of the shovel. A 2 600-lb. hammer was specially cast, having a wedge-shaped cutting edge, about 8 in. deep running parallel to the track. The cutting edge was placed in this way in order that it would not injure the remaining concrete in the street. This scheme worked excellently, and no trouble whatever was found in breaking up the concrete and adhering to the schedule. The concrete breaker, which would swing through 360°, was placed on one track, and on the other track was placed the derrick car, having a 60-ft. boom which would also swing through 360°, and could move itself along the track. The larger pieces of concrete were picked up by the derrick and loaded

Mr. Voree.

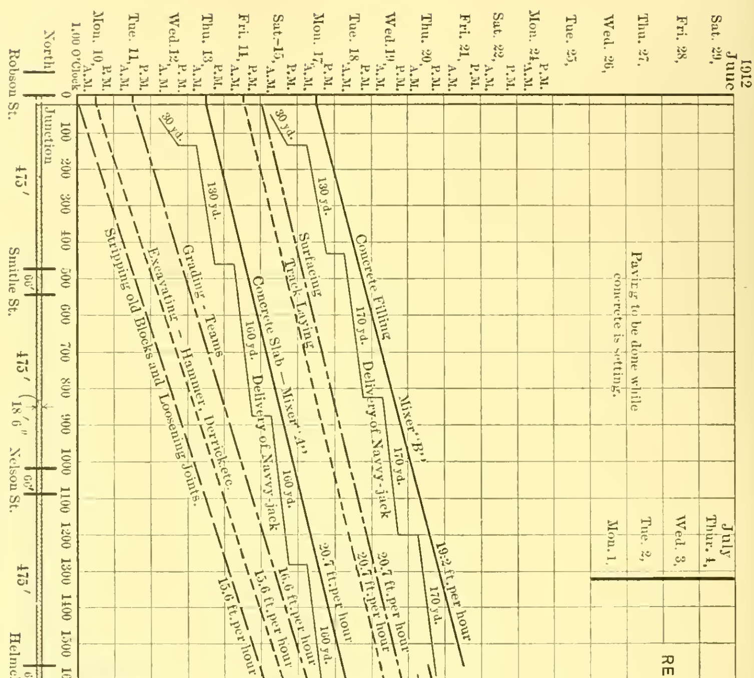

FIG. 5.

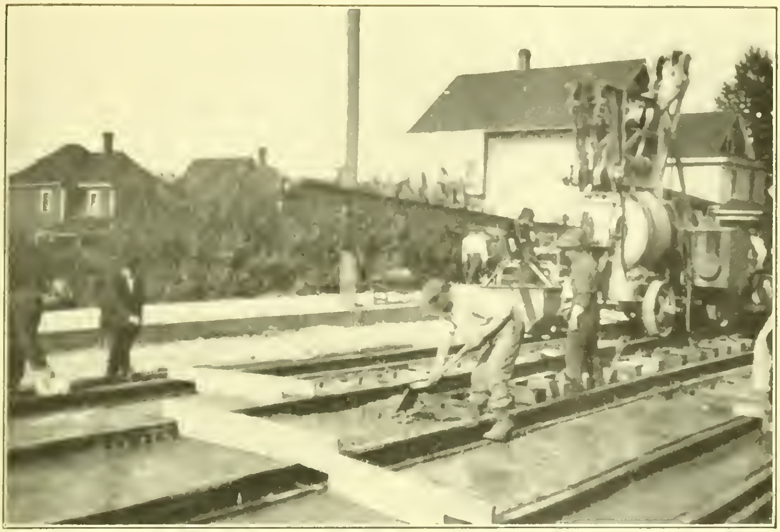

FIG. 1.—BRINGING CONCRETE TO UNDERSIDE OF PAVEMENT, BROADWAY, VANCOUVER, B. C. CONCRETE INSIDE THE RAIL LEFT LOW FOR 6-IN. GRANITE FLANGEWAY BLOCKS.

FIG. 2.—CONCRETE BREAKER, GRANVILLE ST., VANCOUVER, B. C.

on flat cars placed behind the concrete breaker, and the smaller material and earth were loaded in skips and then into wagons standing alongside the excavation (Fig. 2, Plate XL). As soon as the concrete was broken for a rail length, the concrete mixer and derrick car were backed up on the old track and the operation was repeated. Work was carried on in two 10-hour shifts, and the cost was as follows: Taking up wood blocks, 8.9 cents per sq. yd.; tearing up old track, 11 cents per ft. of single track; breaking up concrete, 40 cents per sq. yd.; excavation, loading, and dumping, 82½ cents per cu. yd.; removing material by teams and work train, 65 cents per cu. yd. These figures may seem high, but when one remembers the rate paid for common labor, and that teams are paid $8 per shift of 10 hours during the day, and $9.50 per shift for night work, the writer does not think that the cost was excessive. Mr. Vorce.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE FLOOD OF MARCH 22D, 1912, AT PITTSBURGH, PA.

Discussion.*

By Kenneth C. Grant, Assoc. M. Am. Soc. C. E.†

Kenneth C. Grant, Assoc. M. Am. Soc. C. E. (by letter).—The seventeen reservoirs described in the paper have a combined capacity of about 60 000 000 000 cu. ft. The greatest flood that has ever visited Pittsburgh had a volume of only 26 000 000 000 cu. ft. above the flood stage. The recent flood of January 9th, 1913,‡ the fourth greatest in 48 years, had a volume of only 13 000 000 000 cu. ft. above the flood stage. Mr. Grant.

It is not unreasonable to expect, therefore, even without the graphical proof described, that the reservoir system proposed by the Flood Commission will control floods at Pittsburgh effectively; and the discussions have not expressed any doubts of their effectiveness in this respect. It has been stated, however, that the reservoirs could not be used for flood control and for improving the low-water flow of the rivers as well. It is claimed that such a scheme is utterly impracticable, and has never reached a practical solution.

Reservoirs combining the purpose of flood control with those of water supply, water-power, and navigation are not an experiment. They have been built and are being operated successfully in other countries. The writer has had the opportunity of examining these works on two occasions during the last few years. Some of them he has described briefly§ in his discussion of the paper by John H. Lewis, Assoc. M. Am. Soc. C. E.; others are to be found in Germany and

* Continued from December, 1912, *Proceedings*.

† Author's closure.

‡ Described in *Engineering News*, January 30th, 1913.

§ *Proceedings*, Am. Soc. C. E., for December, 1912, p. 1753.

Mr. Grant. Bohemia. A large reservoir constructed on a tributary of the Oder River, in Southeastern Germany, for flood control and water-power development, has given such good results during the 8 years since its completion, that another and larger project has been constructed, and is just about to be put in operation. Another large project is being constructed by the German Government on a tributary of the Weser, combining the purposes of flood control, improvement of navigation, and water-power development.

One of the discussions calls attention to the conditions that would obtain where the reservoirs were constructed on streams carrying a large quantity of acid from the mines. It is claimed that the greater specific gravity of the acid would lead to its concentration at the bottom of the reservoirs, and that, in the event of a great flood filling the reservoir, this acid would be swept out and become a menace to water supplies below. As a matter of fact, this very property of the acid, which would keep it at the bottom of the reservoir even when overflow took place over the spillway, simplifies the problem of avoiding the conditions of concentrated acidity that are described. By placing gates at the extreme bottom level of the reservoir, and others higher up, this acid could be released gradually from the bottom gates during flood times, to mingle with, and be diluted by, the large flood discharge. If found necessary, a simple arrangement of baffles would make this mixture complete.

The reservoir particularly referred to is near the mouth of Loyalhanna Creek. It is suggested that the dam be built farther up the creek, above Latrobe, in order to avoid storing the mine water, most of which originates on the lower portion of the drainage basin. There are several reasons why this should not be done, chief among which are the great reduction in the drainage area controlled, and the fact that the very conditions pointed out as so objectionable—the flushing out of the acid in the existing pools—would be done away with by locating the dam near the mouth and taking care of the acid as before suggested.

It must be borne in mind, moreover, that only two of the thirteen reservoirs proposed on the Allegheny Basin, those on Loyalhanna and Black Lick Creeks, are to be on streams that carry any appreciable quantity of mine drainage; and that the discharge from the other eleven reservoirs will dilute, to a large extent, any acid water that may come from these streams. Such dilution, of course, would not occur if the rainfall were local around the Loyalhanna or Black Creek basins; but that is just where the beneficial effect of the reservoir near the mouth of each creek would make itself felt, by holding back the acid at the bottom until a favorable time for its discharge.

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PAPERS AND DISCUSSIONS

THE SEWICKLEY CANTILEVER BRIDGE OVER THE OHIO RIVER.

Discussion.*

By Messrs. V. R. Covell and Henry H. Quimby.

V. R. Covell, Assoc. M. Am. Soc. C. E. (by letter).—Referring to Mr. Le Conte's discussion of the type of structure used, the following conditions controlled: Mr. Covell.

The Act of Congress relative to bridges across the Ohio River, approved December 17th, 1872, as amended February 14th, 1883, requires that bridges above the mouth of the Big Sandy shall have at least one channel span with a clear waterway of 500 ft. between piers, measured at the low-water line, and with a clear height of 90 ft. above low water and 40 ft. above local highest water. The demands of the river shipping interests and the ruling of the War Department fixed the span at 750 ft., from center to center of piers. The local conditions were unfavorable for a suspension bridge, and the cantilever was adopted.

Henry H. Quimby, M. Am. Soc. C. E. (by letter).—Two of the features of the subject treated in this paper suggest some experience that may be of interest as a discussion. One is the construction of the floor, and the other is the deformation of the trusses in erection. Mr. Quimby.

The character of the floor construction of such a bridge is not the least important part of the design, because of the special value of lightness as well as of durability. The use of wood blocks is commendable, if the base be substantial. The paper does not state the size and character of the buckles of the floor plating. The thickness of the concrete paving base is said to average 3 in., which probably allows about 2 in. over the flat portions. If the plates are multiple-

* Continued from December, 1912, *Proceedings*.

Mr. Quimby.

buckled, are the ridges or divisions between the buckles depended upon for cross-support, or is transverse stiffening provided?

Buckled floor-plates, as originally used, were in single buckles turned up, forming vaulted arches, and were supported on all four sides. Later, the custom arose of turning the buckles down and using long plates with several buckles in each, and without any cross-beams under the divisions. The strength of such a member is not easy to figure. It depends in part on the narrowness of the divisions, and in part on the thickness, and therefore the distributing value, of the concrete over them. The divisions must obtain support from the arch action of the inclined sides of the adjacent buckles. If a division be wide and have only a thin non-distributing paving over it, the corners or edges of the division will yield successively and alternately down and up under rolling loads. Only a slight movement of this character is required to crack and ultimately crumble cement concrete. When this stage is reached, the pavement is near to destruction.

Such a case occurred recently in the asphalt paving of a new deck on an old bridge where $\frac{5}{16}$-in. plates were used, supported on stringers, 5 ft. 6 in. apart in the clear, buckles, 5 ft. 3 in. by 4 ft. 3 in., and divisions varying from 1 to 12 in. in width because of varying panel lengths of the structure. Considerations of dead load and also of established street elevation demanded a thin floor, and so the paving base was used only to fill the depressions of the buckles, and the asphalt binder coat—1 in. thick—was laid directly on the flat parts of the plates. The wearing surface was 2 in. thick. The vehicle travel was very heavy, and, for several months during the reconstruction of the other half of the deck, it was concentrated on one shoulder in a narrow line. Within two or three months it was observed that the paving was breaking in several places, and it was found that these places were the wider, flat portions of the deck plates. In time the paving at two points was sufficiently pulverized to become dislodged and form holes, exposing the plate. Then the dropping of wheels off the edge of the surrounding paving pounded down one edge of each division thus exposed.

The remedy applied was the placing of chair supports under all divisions—a floor member of the old structure being suitably situated below each point—and then repaving the whole shoulder, the grade, at the same time, being raised $1\frac{1}{2}$ in., so as to have a little concrete over the entire surface of the floor-plates.

The lesson seems to be, that if multiple-buckled floor-plates are used without cross-support, the divisions should be as narrow as practicable, and they should be covered with a bed of concrete, say, as thick as the width of the division.

It would seem that the provision for making the field connections of certain members is not covered in the paper. $L - U_{11} - U_{12} - L_{12}$

form a quadrangle, made rigid by M_{11} — U — L_{12}, diagonally across it. The camber allowances in the lengths of these members must have distorted the quadrangle so much, while the members were without stress, that neither pilot point nor drift would avail to get the diagonal connected up, unless an easement were made somewhere. By what means was this connection made? Mr. Quimby.

In the Red Rock cantilever bridge, where the main span is 660 ft., with a panel similar in character, the writer accomplished the connection of the panel by designing the lower chord splice below it so that it constituted a slip-joint. About 2 in. opening of this joint was required to permit the closing pin of the panel to be driven, and it remained open until the traveler advanced far enough in the erection of the succeeding panels to stretch and compress the truss members sufficiently to close the joint, and then it was riveted up. During the time that the joint was open the splice-plates prevented lateral displacement, and an adjustable bridle took care of possible tension from wind pressure.

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PAPERS AND DISCUSSIONS

PORTS OF THE PACIFIC.

Discussion.*

By H. M. Chittenden, M. Am. Soc. C. E.†

H. M. Chittenden, M. Am. Soc. C. E. (by letter).—Mr. Goodrich has permitted an unnecessary sensitiveness to the writer's rather jocular commentary on his ambitious vision of the future port of Los Angeles to give a false color to his estimate of what the writer said about that port. A careful perusal of the treatment of that subject, mainly in Section II of the paper, will show it to be substantially accurate, though lacking in detail, as was necessarily the case. Particularly inexcusable is the following remark by Mr. Goodrich: "Nothing but an obviously exaggerated newspaper report has been used in describing the port conditions at Los Angeles." The most superficial examination of the paper would show the absurdity of that statement. As a matter of fact, all available literature, including Major Fries' admirable report, was used, and this was supplemented by a long list of questions sent to an official of the Port of Los Angeles asking for data on all phases of the question. Mr. Chittenden.

As to the newspaper report, which is what really annoys Mr. Goodrich, it came to hand just before the paper was finished. It fitted in so perfectly with the writer's treatment of "plans for the future" and seemed to be so much in detail and to bear so unmistakably Mr. Goodrich's authority, that the writer ventured, with some misgiving, it is true, to make use of it. If it committed him to any erroneous suggestions as to the future of Los Angeles which later data would have modified, he is, of course, only too glad to accept corrections.

The writer will not comment on Mr. Goodrich's references to estimates of cost of Los Angeles work, nor to administrative control,

* Continued from December, 1912, *Proceedings*.

† Author's closure.

Mr. Chittenden. for they are so manifestly unjust that Mr. Goodrich must himself realize the fact if he will but compare his criticism with what the writer really said.

The comment on the writer's parenthetical reference to the Los Angeles and San Gabriel Rivers is a case of assuming that something was meant which was not said, and then criticising that something. The writer's description of the natural state of those streams is literally correct; but he did not anywhere say, or even imply, that these streams were a menace to the integrity of the harbor. The writer has carefully looked over his paper to see if it contains any hint of danger "of the inner harbor of Los Angeles becoming silted up by material carried in the floods" of these streams, and he can find none. All this, and the careful refutation of it, are purely the result of Mr. Goodrich's imagining what the writer did not say.

In short, Mr. Goodrich has permitted himself, most mistakenly, to class the writer as a "knocker" of the Port of Los Angeles. Yet a perusal of the paper will show it to be, not only wholly sympathetic in tone, but full of admiration for the wonderful work which the people of Los Angeles are doing.

The impartial reader of the paper cannot fail to observe that it endeavors to draw a just comparison among the leading ports of the coast as to their respective advantages and drawbacks. The partisans of each will naturally take exception as to their particular port, magnifying the advantages and minimizing the drawbacks; but, when all is said and done, the case for the principal United States ports on the Pacific Coast will stand substantially as summarized below:

San Diego.—Advantages: A magnificent natural harbor which, with a moderate degree of development, would accommodate the shipping of the world.

Drawbacks: A sharp restriction of hinterland (tributary commercial territory) by the national frontier immediately to the south and a mountain range on the east. The first natural outlet to the eastward is in the vicinity of Los Angeles.

Los Angeles.—Advantages: Commercially located where the great southern transcontinental routes, and also one from the interior, debouch from the mountains to the coast plain; strategic location for ocean terminal; extensive and productive near-by hinterland; good connection with the hinterland farther back; climate an asset of enormous value in development of country.

Drawbacks: Entire absence of an adequate natural port, and consequent necessity of creating one almost wholly by artificial means; cost of port development greatly enhanced by this fact.

San Francisco Bay.—Advantages: A perfect natural harbor of great magnitude, close to the sea; a hinterland which embraces the heart of California's agricultural and mineral producing regions—one

of the richest in the world; on the line of the great routes north and south; in direct rail connection with all the region to the eastward; a commerce and development half a century old, with the immense force of customs and established lines of trade in its favor; most centrally located of all the ports for the commerce of the Pacific; important naval and military bases. Mr. Chittenden.

Portland.—Advantages: Unexcelled strategic location at great crossroads of north and south, and east and west routes; a productive local hinterland (agricultural) in the Willamette and other near-by valleys; immense timber resources; a water-grade route to the eastward—the only one from the Pacific Coast of the United States; consequently a vast natural interior hinterland.

Drawbacks: Long distance and defective natural channel to sea which discourages entrance of largest vessels—a drawback as yet only partly overcome.

Puget Sound.—Advantages: Almost perfect natural harbors, the principal defect being excess of depth; center of great timber producing region; center of great fish producing region; limited local agricultural hinterland; extensive agricultural hinterland back of the mountains; the natural emporium for the Alaska trade; well located for Oriental trade; important naval and military base.

Drawbacks: Walled off from natural hinterland to eastward by an immense range of mountains, the main passes being less than 75 miles from tide-water and more than 3 000 ft. high; crossed, however, by three railways, which give same terminal rates as to Portland.

A most striking illustration of the drawbacks above described of both the Lower Columbia and of Puget Sound was seen in the conditions which prevailed on January 7th and 8th, 1913. A fierce storm was causing wrecks and great loss of life at the mouth of the Columbia, while deep snow, avalanches, and (soon after) floods from melting snow completely paralyzed rail traffic over the mountains back of Puget Sound, and caused considerable loss of life. Permanently to remove either the handicap of the Columbia Bar or the handicap of the Cascade Mountains will cost an enormous sum, yet both ends must be classed as within the range of practical engineering.

In the foregoing comparative estimate of advantages and drawbacks of Pacific Coast ports it will be noted that San Francisco stands first; and it seems to the writer that she will long continue to stand first, because of her unexcelled advantages. It has also become more and more apparent to him that, unless ports are close competitors in the same field—not distant competitors like Puget Sound and California ports—a mere superiority of port facilities cuts little figure in the matter of competition. Vessels are bound to go where the trade is, even if docks are inferior and charges high, and the writer does not believe that San Francisco or Seattle will take much trade from

Mr. Chittenden. each other by reason of superior expenditures on docks and wharves; but, with two nearby ports like Seattle and Tacoma, both of which are equally well located, as far as local trade is concerned, and between which the vessel owner sees no choice except in the matter of conveniences and charges, a superior equipment in either might be decisive in its favor.

The writer will refer to one other point, that of port administration. Mr. Goodrich says that Los Angeles has corrected the undesirable condition in port administration prevailing at the time this paper was written. That is fortunate, but the writer does not believe that anything approaching an ideal system will ever be worked out as long as port administration is a function of city government and is thereby necessarily involved in city politics and subject to the vicissitudes of changing administration. That this is also the view of sagacious observers on the ground is evident from the following extract,* received after the foregoing statement was written:

"Inefficiency and delay, bickering, obstruction, and even deadlocks will unquestionably be the rule rather than the exception so long as the present machinery for harbor development is relied on by the city. The concentration of power, authority and responsibility in one board, under which the legal, engineering and administrative departments would be placed, will probably have to come before much satisfactory progress is made on this all-important municipal project—the development of Los Angeles harbor."

In a most instructive pamphlet recently issued by the Commonwealth Club of California, Professor C. T. Wright, of the University of California, made some pertinent observations on port administration. The following are some of his conclusions:

"The organization of the harbor governing body should be simple and efficient. Its members should be few in number, with a tenure of office sufficiently long to permit of the formation and inauguration of a consistent policy.

"The appointment of a full new board almost always results in an entirely new policy. A new board usually rejects a great part of the programme outlined by its predecessors, and spends a large part of its time in preparatory work before it begins or pursues any real constructive programme; and this in turn is superseded by the plan of a succeeding board. The harbor governing body, if ideally constituted, would be a perpetual body, a certain proportion of whose members would retire each year. In other words, it ought not to be possible, or at least it ought not regularly to occur, that the entire board should retire at one time, or even that a majority of the board should do so.

"The harbor governing body should have full authority to carry out the work which it exists to do, subject of course to review by the courts or other higher authority. This power of the governing board should extend to all activities which directly concern shipping.

* From the *Los Angeles Tribune*, January 17th, 1913.

"A harbor governing body should represent a constituency of ample financial ability." Mr. Chittenden.

These are sound fundamental principles. While it is perfectly true as a rule that "What'er is best administered is best," and that a very efficient law inefficiently administered may give poor results while a defective law efficiently administered may give good results, still, in the long run, the best law will give the best results. The writer believes that the Washington State statute comes nearer to satisfying the above fundamentals laid down by Professor Wright than any other. The port organization is simple, consisting of three members, entirely free of State, county, or city politics, and responsible directly to the people of the Port District.

The harbor governing body (Port Commission) is a continuing body, the term of office being three years, and only one member being elected each year. No sudden change of policy with its disturbing features is possible, as is the case with the San Francisco Board, which is liable to change bodily with every change of State administration.

The powers of the Port Commission are extensive; its activities are directed to the single purpose of port development; it is almost supreme within its restricted sphere, and its authority satisfies perfectly Professor Wright's requirement that it be "both extensive and intensive."

The territorial extent of the constituency and also its financial ability are determined in the original organization of the district. The Port of Seattle, for example, was made co-terminous with King County, in which the City of Seattle is located. "Ample financial ability" does not necessarily mean large financial ability. A small port may have an organization and a financial ability "ample" for itself, which might be quite inadequate for a great port. The system is thoroughly elastic, and is applicable to all conditions, and the writer would commend a study of it by port authorities wherever changes are contemplated in port organizations.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

TUFA CEMENT, AS MANUFACTURED AND USED ON THE LOS ANGELES AQUEDUCT.

Discussion.*

By Messrs. Rapier R. Coghlan and Charles H. Paul.

Rapier R. Coghlan, Esq.† (by letter).—The blended cement manufactured by the City of Los Angeles for use in the Aqueduct would have been spared many criticisms if a more fortunate name had been applied to it. The officials of the Aqueduct Commission, in their various papers concerning the product, have used indiscriminately the terms, tufa-cement, tufa-Portland cement, puzzolan-cement, and puzzolan-Portland cement. Some engineers, not aware of the real composition of the cement, have claimed for it many of the bad qualities of the artificial puzzolans, such as are manufactured from blast-furnace slag and hydrated lime; while others have claimed that it is a simple adulteration of Portland cement with clay or clay-like material, and therefore harmful. These claims are widely at variance with what is actually the case. Mr. Coghlan.

The term sand-cement, while ordinarily meaning a blend of pure silicious sand and Portland cement, is not of necessity restricted to such a combination, but can be applied to a blend of Portland cement with granitic, basaltic, or many other sands that contain other elements and minerals than quartz. Even in the case of the Aqueduct cements, the tufa had to be reduced to a coarse sand before it was possible to blend it with the Portland cement. For this reason, it would seem that the best name to adopt for such a blended cement would be sand-cement. The nomenclature allows of the use of many materials, which have been experimented with and found to be as well adapted to the process as tufa.

Experiments have been conducted by the U. S. Reclamation Service on the blending of Portland cement with basalt, basaltic tufa, rhyolitic

* Continued from January, 1913, *Proceedings*.

† Mfg. Cement Chemist, U. S. Reclamation Service Elephant Butte, N. Mex.

Mr. Coghlan. tufa, granite, andesite, trass, infusorial earth, and sandstone. In every instance it has been found that excellent grades of sand-cements can be produced from these materials, cements, which, up to the present time, have shown little or no decrease in strength. Researches on limestones and quartzites have shown, that, excepting those instances in which the amount of material added to the Portland cement was less than 40% by volume, these materials are not suited to the process.

It is a geological fact that the ultimate products of the decomposition of igneous or volcanic rocks are clays. Experiments by eminent men in the Engineering Profession have shown the deleterious effect of the addition of clay to Portland cement. Some concrete failures have been traced to the presence of too much clay in the sand or gravel, so that engineers have become accustomed to setting a maximum limit on the quantity of clay allowable in sand which is to be used for concrete. The results obtained with the so-called tufa-cement do not indicate that clay or clay-like substances have been blended with Portland cement. Tests made by the U. S. Reclamation Service laboratory, on material from the Aqueduct plants, corroborate fully the data set forth by Mr. Lippincott; in fact, by using a brand of cement (which was a recognized standard brand) other than that used at the Aqueduct plants, the results were greatly exceeded.

Careful chemical analyses of the raw materials suitable for sand-cement manufacture show that they are all dependent for their fitness on the quantity of colloidal or active silica; and many of the materials tested by the Reclamation Service experimentalists show a higher content of colloidal silica than the Los Angeles tufa. Most of these materials are harder to pulverize than tufa, but the resultant cement has proven to be as good in every particular. So fruitful have been the results of the researches that three sand-cement plants are being operated or erected by the Service: at Arrowrock, Idaho, Lahontan, Nev., and Elephant Butte, N. Mex. When all three of these plants are in operation, they will have a combined daily output of 2 300 bbl. of sand-cement. Such a wholesale manufacture and use of sand-cement would not be permitted by the engineers in charge of the work if the product was considered unreliable or apt to deteriorate with time. At those places where the cement is being used it is proving its worth. Little or no difference is apparent between sand-cement concrete and Portland cement concrete. It must be mixed a little dryer, and it sets and hardens more slowly than Portland cement concrete. The forms come away clean from the set concrete, and the water that comes to the surface brings little or no cement with it. Daily tests are made on all concrete placed in the dams, and the results of the compressive tests show that it is amply strong for the great stresses put upon it.

Sand-cement is of economic importance to the engineer in the West, as it allows him to use a cheaper cement at a price a little more than half of that of straight Portland. Before undertaking its manufacture or use, two things must be settled: The Portland cement must be of undoubted quality, a standard brand passing all tests; and the blending material must contain colloidal silica and be in an almost unaltered condition. Mr. Coghlan.

Charles H. Paul, M. Am. Soc. C. E. (by letter).—This paper opens up the subject of the use of blended cements, which is of great interest to all engineers engaged in work in the West, where high freight rates and long wagon hauls usually combine to make the cost of Portland cement excessive. Mr. Paul.

In the construction of the Arrowrock Dam—which is being built by the U. S. Reclamation Service to store the flood waters of the Boise River—about 550 000 cu. yd. of concrete will be laid, and the cost of cement is, of necessity, a most important item. The dam is about 22 miles above the City of Boise and 17 miles above Barberton, the nearest point on the Oregon Short Line Railroad. A railroad from Barberton to Arrowrock has been built by the United States Government, over which the freight rate on cement charged against the work is 23 cents per bbl. The commercial freight rate on cement from Utah mills to Barberton is $1.14 per bbl., from California points $2 per bbl., and from Kansas points, $2.09 per bbl.; so that the total freight charges on cement to the Arrowrock work are from $1.37 to $2.32 per bbl.

Laboratory tests on sand-cement have been carried on by the Reclamation Service for 6 years or more, and, during the winter of 1910-11, when the investigations in connection with the storage works on the Boise River were nearly finished, a series of these tests was started, using for blending material the native rocks in the vicinity of Arrowrock, particularly granite from the spillway site, a large quantity of which would have to be wasted during the construction period.

TABLE 11.—Average of Three Long-time Tensile Tests on Sand-cement Mortars (1 to 3, by weight, standard sand), Each Test being an Average of 5 Briquettes; and Comparison with Mortars made from Portland Cement used in the Manufacture of the Sand-cements and Tested under the Same Conditions.

Kind of cement.	7 days.	28 days.	3 months.	6 months.	1 year.	2 years.	3 years.	5 years.
Sand-cement	219	333	397	423	457	423	449	444
Portland cement	266	400	425	482	460	496	428	404

Blending material for sand-cement was river sand, bank sand, or granite, blended in each case with equal parts by weight of Portland cement.

Mr. Paul.

TABLE 12.—AVERAGE LONG-TIME TENSILE TESTS ON 13 BRANDS OF PORTLAND CEMENTS USED BY THE U. S. RECLAMATION SERVICE DURING THE LAST FIVE YEARS.

Mortar briquettes, 1 to 3 by weight, standard sand.	7 days.	28 days.	3 months.	6 months.	1 year.	2 years.	3 years.	5 years.
	260	373	448	447	432	411	379	398

It is noticeable that there is not the marked decrease in strength in the case of the sand-cement briquettes as is shown here, and is so common with Portland cements.

TABLE 13.—SAND-CEMENT TESTS ON MATERIAL FROM ARROWROCK DAM SITE, ARROWROCK, IDAHO.

Blending material, spillway granite. Mix, 1 to 3 by weight with standard sand. Strength, in pounds per square inch.

TENSILE TESTS ON STANDARD BRIQUETTES.

Percentage of blending material.	7 days.	28 days.	3 months.	6 months.	1 year.
0	343	428	461	467	408
30	374	417	467	499	494
40	368	467	504	....	....
50	330	419	460	452	410
60	294	369	400	....	....
70	156	237	266	320	335

COMPRESSION TESTS ON 2-INCH CUBES.

Percentage of blending material.	7 days.	28 days.	3 months.	6 months.	1 year.
0	1 077	1 969	3 720	3 744	....
30	1 631	2 454	3 171	4 392	4 656
50	1 471	1 763	2 729	3 121	3 712
70	458	938	1 215	1 256	1 560

The first column of Table 13 (percentage of blending material) in each case represents the Portland cement from which the sand-cement is made, tested under the same conditions.

Investigations of the actual use of sand-cement in structures which had been in service for 10 or 15 years, further tests in the local laboratory, and estimates of cost of installation and operation of the necessary equipment, finally led to the decision to use sand-cement, in so far as practicable, in the construction of the Arrowrock Dam, and to use for blending material granite from the spillway excavation.

A sand-cement plant, with a capacity of 1 000 bbl. per 24 hours, consisting of a crusher and sand rolls, rotary dryer, ball mill, mixing

machine, and three tube mills, all electrically operated, with the necessary bins, hoppers, and conveying machinery, has been erected and has been in operation for about 2 months. The cost of this mill, complete, was about $46 000, itemized as follows: Mr. Paul.

Excavation	$1 500
Foundations	3 750
Erection of building, chutes, etc	8 150
Equipment, including freight	23 000
Installation of equipment	7 850
Electrical work	1 750
	$46 000

The total output of the mill to date has been about 25 000 bbl., and about 20 000 cu. yd. of sand-cement concrete have been placed in the dam, up to the present time.

All incoming Portland cement is tested and accepted before shipment. All sand-cement is tested for fineness, blend, setting time, and tensile strength in 1:3 sand briquettes, and is subjected to the boiling test before being used in the work. Tests for fineness and blend are made every hour that the mill runs, or oftener if either needs correction, and the other tests are made on an average sample from each day's run. The blend now being used is 45% granite by weight, and the requirement for fineness is at least 90% passing a 200-mesh sieve. Standard Portland cement requirements are satisfied by the other tests, and no sand-cement is used until it has passed the 7-day tensile tests of 200 lb. per sq. in. The writer's experience with sand-cement confirms that of the author with tufa cements, that no sample tested has ever failed to pass the boiling test.

TABLE 14.—Mill-run Tensile Tests on Sand-cement Manufactured at Arrowrock.

All tests on 1:3 standard sand briquettes. Strength, in pounds per square inch.

Average of all (52) 7-day tests	258	Average of all (29) 28 day tests	332
Average of 10 lowest 7-day breaks	221	Average of 10 lowest 28-day breaks	304
Average of 10 highest 7-day breaks	297	Average of 10 highest 28-day breaks	366

None of the sand-cement tested has failed to pass either the 7-day or the 28-day requirements. Occasionally, briquettes have been broken at the age of 4, 5, or 6 days, and in every case yet tested these briquettes have shown a strength exceeding 200 lb. per sq. in.

A representative cost of manufacturing 1 bbl. of 45% by weight blend of sand-cement at Arrowrock is given in Table 15, which in-

Mr. Paul. cludes depreciation, on the plant and installation, at a rate which will wipe out the total cost at the time that a total output of 500 000 bbl. is reached. It does not include sacking, as most of the sand-cement will be used in bulk.

TABLE 15.—Cost of Manufacturing Sand-cement.

Items	Unit cost of sand-cement, per barrel.
Granite delivered to crushers	$0.02
*Portland cement, including freight and storing	1.35
Handling and storing Portland cement	0.08
Labor, operating	0.10
Power and lights, including maintenance, etc	0.16
Installation, depreciation, supplies, repairs, etc	0.14
Total cost	$1.85

* Portland cement @ $2.36 per bbl., f.o b., Arrowrock.

The use of sand-cement under actual working conditions brings out several characteristics that are different from those of Portland cement.

Comparing sand-cement concrete with Portland concrete, batch by batch, an experienced concrete man will note no difference, either in its appearance or in its behavior while being spread, but, after a run of an hour or two, in mass work, it will be noticed that an ordinary plastic or quaking mixture gives off water more readily than does Portland cement concrete, and that this water is clearer and apparently carries less cement than is the case with ordinary concrete.

A mushy or really wet mixture which is worked up continually shows, at the surface, an accumulation of sloppy material which is more marked than in Portland concrete and sets and hardens very slowly. In fact, it is the writer's opinion that, for this reason, extremely wet mixtures should be avoided, especially with sand-cement concrete.

Sand-cement concrete sets more slowly than ordinary Portland concrete, and its slowness in hardening is very marked. It is natural to assume that internal shrinkage stresses, due to setting and hardening, will be less on this account, but, at the present time, we have no other evidence on this point.

The writer has not noticed that sand-cement concrete adheres to the forms any more than Portland concrete, and as he has had no occasion to use sand-cement plaster, he has not had an opportunity to compare it with ordinary cement plaster.

The author states that tufa cement makes a denser and more impervious concrete than Portland concrete. The writer has heard this statement made before, both in connection with tufa cement and

sand-cement, but has never found record of any experiments or definite evidence to that effect. It would be interesting if the author would discuss that point more fully, and give the members the benefit of any experiments along that line with which he is familiar. Mr. Paul.

It is natural for the ordinary construction man to look on a new product with prejudice, especially when it shows characteristics to which he is not accustomed; but, supposing, for the sake of making a comparison from another viewpoint, that sand-cement had been in general use for years and that Portland cement were the new product being introduced; would we look with favor on a cement which almost invariably showed a decrease in strength after a few months? Or one which not infrequently failed to pass the boiling test? Would we consider it a favorable sign that the excess water coming to the surface of concrete apparently brought with it an unusual quantity of cement? Quick-setting cements are usually considered undesirable for use in mass work, and were we accustomed to the slow-setting and slow-hardening sand-cement, might we not look with suspicion on a quick-setting cement, especially if one of the quicker-setting Portlands happened to be the first of the new product to fall into our hands?

The use of sand-cement in mass work, where the requirements are enough to justify the installation of the necessary grinding machinery, where suitable blending material is available, and where the transportation charges on Portland cement amount to a considerable portion of its cost laid down, will result in a marked saving in construction costs, and will give a product which is at least the equal of the Portland cement from which it was made, in fact, one which, for ordinary requirements, is not open to the least suspicion.

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PAPERS AND DISCUSSIONS

SPECIFICATIONS FOR METAL RAILROAD BRIDGES MOVABLE IN A VERTICAL PLANE.

Discussion.*

By Charles H. Mercer, M. Am. Soc. C. E.

Charles H. Mercer, M. Am. Soc. C. E. (by letter).—The writer is connected with a company which fabricates and erects a considerable number of bridges movable in a vertical plane, and, after consulting freely with the electrical and mechanical engineers of this company, has considered it appropriate to compile and present the results, with the hope that they will be found to contain, at least in part, sufficient merit to commend themselves to Mr. Leffler. Such a set of specifications can best become a smooth working document by criticism followed by a broad review of the whole problem. Mr. Mercer.

Paragraph 2.—The "Specifications of the New York Central Lines for Steel Railroad Bridges," for 1910, are made a part of the author's specifications. These specifications require the railroad company to furnish and lay the track, rails, timber floor, etc. This is logical for an ordinary steel bridge, but does not seem to be proper for a counterweighted bridge. The contractor for a bridge of this character cannot balance the bridge until the floor is laid. If the railroad does this work, the labor forces will be mixed, which is objectionable. Besides, the majority of these bridges are erected open, which tends to increase the complications if the railroad company lays the track. The specifications would be more harmonious if they required the contractor to do this work.

Paragraph 21.—The constants given in this paragraph do not seem to be sufficiently specific; thus, for rolling friction, the radius of the roller and whether one or two faces are in contact should be considered. The writer has had to do with "movable" bridges moving on rollers of 12 and 100 in., respectively, and with segmental rollers of

* Continued from December, 1912, *Proceedings*.

Mr. Mercer. much larger diameter. Although there is no agreement on the constant used in the several formulas for rolling friction, it would seem that the maximum of these could be used advantageously in ascertaining the resistances to be overcome.

The loss caused by the stiffness of wire cables depends on the sheave diameter, the number and angle of the bends, and the type of rope. The usual limitation of sheave diameter is not made in these specifications, probably because the stresses due to bending of ropes over them are taken into account. It would seem, however, that the sheave diameter should be considered in calculating the power requirements.

Paragraph 22.—The coefficient for journal friction is given as 0.07. The writer thinks this should be covered by Paragraph 21, using the same constant as for friction on trunnions, that is, $\frac{1}{8}$ for starting and $\frac{1}{16}$ for running.

Paragraph 23.—This paragraph reads: "The time to open the bridge after the ends are released shall be as specified on the proposed drawing."

The writer suggests that it be changed to: The time to open the bridge after the ends are released shall be approximately as specified on the drawings.

The writer has had to do with contracts for two bridges under these specifications with the company represented by Mr. Leffler, and there has been no thought of taking exception to this paragraph, nor need there be when the specifications are interpreted broadly by a reputable engineer; but the specification is now apt to be more widely adopted and made a part of contracts. In such a case, could not an unscrupulous representative of a buying company make the contractor suffer if the bridge required somewhat more time to open than was specified? Bascule and lift bridges are not designed by contractors, but ordinarily by the patentees, who are experts for their respective structures, and it is a proper assumption that they are more expert on these special bridges than the contractor's corps of engineers. The specifications and plans on which the contractor bids are complete, specifying in detail the requirements of the prime mover, the size of all pinions, gears, shafting, etc., and the sectional areas of all the component members, as well as the details for their connection. The contractor is told just what to do. Except for workmanship and materials, it does not appear to be equitable that he should be held responsible for results. A contract based on divided responsibility is wrong in principle; let the designer be responsible for the design and the contractor for the construction and materials. The writer feels that, in specifications of this character, the portion covering design should be separate, and the contractor should not be responsible for it if designed by any other than himself.

Paragraph 36.—The writer believes that trunnions bearing on phosphor-bronze should be of axle steel rather than structural steel. The harder steel would be less apt to cut and score. Mr. Mercer.

Paragraph 126.—This paragraph provides for tests of trunnions on their bearings, and the test requires special apparatus, with consequent delay and expense. The paragraph might well be omitted, but if there is a demand for some such test, the results sought could be attained by placing the trunnions in their respective bearings on a surface plate.

Paragraph 156.—This paragraph states that: "All machinery parts shall be designed with sufficient strength to resist the greatest pressure which can be exerted by the motor." Paragraph 173 specifies an overload for direct-current motors of 200% for 3 min., and 400% momentarily. Apparently, this overload for the motors is simply a test of the insulation, but, taking the requirements of Paragraph 156 literally, the machinery would have to be designed for an overload of 400% with the units specified in Paragraphs 133 and 135, which results would be abnormal.

Paragraph 168.—The maximum piston speed of the gasoline engine is lower than required by standard practice. In all cases the piston speed should depend on the size of the engine. The advisability of operating the locking machinery by the gasoline engine is doubted. If operated by hand, the machinery would be much simplified, and there would be much less likelihood of jamming the lock machinery when undertaking to drive home when not in proper position.

Paragraph 169.—"Water-proof" should be changed to "weather-proof," as it is better to have a semi-open type of motor properly housed. In the case of a totally enclosed motor, the operator seldom inspects it as thoroughly as he would if he could examine the working parts without removing the cover.

Paragraph 172.—The words, "direct-current," in the first line should be omitted.

Paragraph 173.—It would be much clearer if, for each bridge, a definite statement was made as to how much torque the motor had to develop in foot-pounds for 3 min., and the exact time the motor would have to carry the normal torque of 400 per cent. These values should then be selected so that a standard motor which fulfills one will fulfill the other without a special arrangement of the winding.

The percentage of drop in voltage which will be allowed under full-load conditions, between the switch-board and the motor terminals, might be stated for each proposition.

Paragraph 175.—This paragraph is applicable to a back-geared motor. It may not be advisable to use such a motor in every case.

Paragraphs 181 and 182.—These paragraphs are slightly confusing. Paragraph 181 calls for a controller for each motor. Paragraph 182 calls for series-parallel control, which can be applied to two or more

Mr. Mercer. motors. It might be better to have the control specified for each case, because, where the operating machinery does not rotate with the bridge, it would seem as if a remotely controlled contact or board could be used to advantage, especially if the operator's house is some distance from the bridge.

Paragraph 188.—The writer believes the motor-operated, emergency brake to be too complicated for the average operator. A better brake, as far as concerns reliability, less complication, and being more nearly balanced in all positions of the bridge-leaf, is obtained with the same type as that used for the operating motors, provided the size required does not exceed the largest size manufactured. In case it does, two such brakes should be used.

Paragraph 190.—It seems to be desirable that the emergency-brake switch be in closed position, thus releasing the brake before power is available for the operating motors. In this case the switch would be electrically interlocked, but the operator would be free to apply the emergency brake at any time, regardless of the position of the controller arm.

Paragraph 192.—Submarine cables, if needed, should be furnished and laid by the contractor for almost the same reasons as those given for laying the floor, and to obviate division of responsibility due to possible delay. The trench for such cables if required, might be dredged or dug by the railroad company.

Paragraph 193.—For bridges moving in a vertical plane, the logical place at which to deliver the current to the electrical contractor is at the switch-board.

Paragraph 197.—If the electrical equipment is installed as required in these specifications, there seems to be no need of requiring everything to be in accordance with N. E. C. Standard, as this standard is devised for installations where the danger of fire is great, which is not true on a bridge. Bridges were not considered in making up the N. E. C. Standard, and its application to them is not sufficiently definite.

Paragraph 212.—In measuring currents, it seems to be standard practice to use shunts for the ammeter, thus avoiding heavy currents through the measuring instrument and the consequent impairment thereto.

Paragraph 217.—The type of the alternating-current motor will depend on the kind of current available, and need not be a 25-cycle, 220-volt, three-phase type.

Paragraphs 265 and 266.—Phosphor-Bronze.—It is well to preface any discussion on this subject with a statement of the fact that it is most difficult to obtain consistent physical results from any given phosphor-bronze alloy, and the most expert superintendence in the foundry is required to produce uniform results.

Probably on account of the comparatively small quantity of phosphor-bronze used in movable bridges, it has not received the attention from bridge engineers that its importance warrants. In looking over bridge specifications in which the composition for phosphor-bronze is given, the writer finds that almost all specify that given in Paragraph 268 of the author's specifications, namely: Mr. Mercer.

Copper	79.7	per cent.
Tin	10.0	" "
Lead	9.5	" "
Phosphorus	0.8	" "

and this composition will be referred to hereinafter as "standard."

Practically all the bearings in movable bridges may be characterized as high-pressure and slow-speed, as against low-pressure and high-speed. For the latter conditions, the "standard" composition seems satisfactory, and specimen tests will give physical results practically as follows:

Compression:

Elastic limit, in pounds per square inch	13 000 to 15 000
Permanent set from 100 000-lb. load, in inches	0.27

This elastic limit is based on a set of 0.001 in. in 1 in.

Tensile:

Yield point, in pounds per square inch	20 000
Ultimate, in pounds per square inch	25 000
Elongation, percentage in 2 in	4
Reduction	3 per cent.

After discussing this subject thoroughly with a leading metallurgist, the writer submits the following composition for high pressure and slow speed:

Copper	85.2	per cent.
Tin	14.0	" "
Phosphorus	0.8	" "

The approximate physical results from this composition on a cylinder with an area of 1 sq. in. and 1 in. long, should be:

Compression:

Elastic limit, in pounds per square inch	20 000 to 22 000
Permanent set from 100 000-lb. load, in inches	0.12 to 0.16

In tension tests, this composition will show some ductility.

The presence of lead in bridge bearings subjected to heavy pressures is objectionable, for the following reasons: Lead does not alloy with the copper-tin bronzes, but is mechanically admixed, and, being a very soft metal, it immediately lowers the elastic limit in compression,

Mr. Mercer. causing the bronze to become plastic and flow under pressure. It likewise, being a soft metal, tends to grip or seize the shaft under conditions of heavy pressure and poor lubrication. The harder the bronze the less likely it will be to seize the steel shaft, providing, of course, the bronze does not exceed the steel in hardness.

Experiments made by the late C. B. Dudley,* M. Am. Soc. C. E., indicate that the relative wear of a copper-tin alloy is one and one-half times that of the "standard" copper-tin-lead alloy.

For convenience, Table XCIII, showing the results of these experiments, is here quoted.

"TABLE XCIII.—Relative Wear of Bronze Bearings (*Dudley*).

Alloy tested.	Composition.					Relative wear.
	Copper.	Tin.	Lead.	Phosphorus.	Arsenic.	
Standard lead (phosphor) bronze S	79.70	10.00	9.50	0.80	....	1.00
Ordinary bronze	87.50	12.50	....	....	....	1.49
Arsenic bronze, A	89.20	10.00	....	....	0.80	1.42
Arsenic bronze, B	82.20	10.00	7.00	....	0.80	1.15
Arsenic bronze, C	79.70	10.00	9.50	....	0.80	1.01
Bronze K	77.00	10.50	12.50	....	....	0.92
Bronze B	77.00	8.00	15.00	....	....	0.86

This table was determined by placing a "standard" and an experimental bearing on opposite ends of the same axle on locomotive tenders. From time to time they were removed and weighed and the wear was recorded. The copper-tin alloy also gave some trouble from heating. These experiments were conducted about 1892.

These results as to relative wear were confirmed by Clamer about 1903, and are set forth on page 426 of the book:

"He used a specially designed friction-testing machine, with test-bearings, measuring 3½ inches x ½ inch. In each experiment the journal (3¾ inches in diameter) made 100 000 revolutions at a speed of 525 revolutions per minute, the same pressure (1 000 lbs. per square inch), oil, and method of lubrication being used throughout. It was found that the rate of wear diminished, though the friction and temperature increased, as the percentage of tin in the bronze was decreased and the percentage of lead increased (see Table XCIV)."

It is to be particularly noted from this table (XCIV) that the friction on copper-tin-lead alloys is approximately 50% greater than on copper-tin alloys, which approximate the writer's composition for high-pressure and slow-speed bearings.

The results, as set forth in the Clamer's Table XCIV, were confirmed later by Spare, from experiments made on the same friction testing machine, namely, the Cornell machine.

* "Lubrication and Lubricants," by Leonard Archbutt and R. Montford Deeley, p. 425.

Mr. Mercer.

"TABLE XCIV.—RELATIVE FRICTION AND WEAR OF LEAD-BRONZE BEARINGS (*Clamer*).

	Copper.	Tin.	Lead.	Friction, lbs.	Temperature above room, °F.	Wear, in grms
1	85.76	14.90	.....	13	50	0.2800
2	90.67	9.45	.....	13	51	0.1768
3	95.01	4.95	.....	16	52	0.0776
4	90.82	4.62	4.82	14	53	0.0542
5	85.12	4.64	10.64	18½	56	0.0380
6	81.27	5.17	14.14	18½	58	0.0327
7	75. ?	5. ?	20. ?	18½	58	0.0277
8	68.71	5.24	26.67	18	58	0.0204
9	64.34	4.70	31.22	18	64	0.0130

The conditions under which the experiments were made by Dr. Dudley, and Messrs. Clamer and Spare differed widely from those which exist in a bearing with a load of, say, from 1 600 to 3 000 lb. per sq. in. and supporting a trunnion which revolves only through 80° during the operation of the bridge. The amount of wear on the copper-tin alloy bearing with 100 000 revolutions, as reported by Clamer, was not appreciable from a practical standpoint. With an average of ten openings per day, for 250 days per year, it would take 40 years for 100 000 operations of a bridge. Therefore, the problem for high pressure and slow speed is entirely different from that covered by the experiments. It has not been overlooked that the frictional area for a certain number of revolutions varies with the size of the journal or trunnion.

It seems obvious that, for high-pressure and slow-speed bearings, stiffness combined with a low frictional resistance is essential, and the wear is unimportant. Stiffness is shown in the composition submitted by a high elastic limit in compression, while sufficient ductility is retained.

To summarize: the writer recommends for movable bridge bearings a copper-tin alloy with a maximum content of 1% of phosphorus (other alloys in excess of one-half of 1% will not be permitted), with physical results specified as follows:

Compression:

Elastic limit, in pounds per square inch. .19 000 to 23 000

Permanent set from 100 000-lb. load, in inches .... 0.12 to 0.16

Tension:

The yield point, ultimate, and elongation in 2 in. to be recorded.

The compression test should be made on a cylinder having an area of 1 sq. in. and 1 in. long and the tension test on a specimen having a diameter of ½ in. The compression elastic limit should be determined

Mr. Mercer. by the load that gives a permanent set of 0.001 in. It may be noted that the physical tests proposed by the writer are nearly a mean between the physical requirements of Paragraph 266 and the physical results which may be expected from the "standard" composition given in Paragraph 268.

The writer acknowledges his indebtedness to Mr. C. R. Spare, Holmesburg, Philadelphia, Pa., for information on the subject of phosphor-bronze.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

PREVENTION OF MOSQUITO BREEDING.

Discussion.*

BY DR. RALPH H. HUNT AND MESSRS. ROBERT A. RUTHERFURD, KENNETH ALLEN, AND W. E. BRITTON.

RALPH H. HUNT, M. D.†—The speaker is sure that no one is more fit or better equipped to speak with authority on this subject than Mr. Miller, for in New Jersey he is called the "father" of mosquito extermination and prevention of mosquito breeding, in that he was the first to make of it a live issue and bring to the notice of the people the fact that the mosquito could be placed under control and rendered less of a nuisance. Dr. Hunt.

Mosquito work began some years ago, under the efforts of an English Army surgeon, Sir Ronald Ross, who demonstrated that only through the mosquito, in this case a variety of *Anopheles,* could malaria be carried from man to man, and that it was the only known carrier of the infection. Previous to that time it had been supposed that malaria was caused by a miasma or by something in the atmosphere, the idea of the disease being very indefinite until, through the investigations of Italian observers, the organism was discovered in the blood of the human being. Shortly after that Sir Ronald Ross demonstrated conclusively that the mosquito was the carrier of malaria. This was the first thing that brought to the attention of the people the necessity of getting the mosquito pest under control. It is seen, therefore, that in those days it was purely a health problem.

Later, the United States Army surgeons in Havana, Cuba—three of a commission of four sacrificing their lives in the investigation—demonstrated the fact that a certain variety of mosquito, *Stegomyia fasciata,* was the carrier of yellow fever. That was proven beyond a

* Continued from January, 1913, *Proceedings*.

† President, Essex County Mosquito Extermination Commission.

Dr. Hunt. doubt; and two things stand out in scientific medicine and health problems to-day, namely, that if there were no mosquitoes, there would be no malaria and no yellow fever.

The malaria problem would be done away with entirely by the extermination of the variety of *Anopheles* which carries the disease; but, the habits of that mosquito being more difficult to control, it has been less controlled, even where it is a menace, than the yellow fever mosquito in yellow fever countries.

The control of the yellow fever mosquito is comparatively easy, owing to its habits of breeding. It seems that it breeds only in barrels and other small receptacles around houses. One has only to prevent water from standing in these receptacles in order to get rid of the yellow fever mosquito. A complete demonstration of the efficiency of work of that sort has been given in Havana, Cuba, a place formerly pestered with yellow fever but to-day absolutely free from the disease. Yellow fever formerly was a positive menace, and retarded the industrial development of the city; to-day there is no more quarantining of the port, and no more shutting up of business.

We know that the Isthmian Canal could probably never have been completed had not this scientific question been met prior to commencing the mechanical work. In the Canal Zone to-day there is no yellow fever, and mosquito control alone has accomplished this wonderful result.

Referring to the speaker's section of the country, namely, Essex County, New Jersey, which place has been pestered from the time it was first colonized, in early days, to the beginning of the present season, we have reports from Colonial times stating that it was almost impossible to live in certain parts of this section of the country on account of the mosquito pest. The Essex County Mosquito Extermination Commission holds that it has faced a difficult problem and has worked it out to a practical solution for the first time, as far as the speaker knows, in the history of the country.

In previous years local boards of health had attempted to control the mosquito, and through them a desultory movement began. Mr. Miller, who was in this work at its inception, states that it began about eleven years ago, and the speaker assumes that that was about the time when the first practical work was attempted by local boards of health with respect to mosquito control. Up to 1912, it was found that such a method was impractical. The local boards of health could not obtain, or would not spend, the money, and even those that did make an effort to do their share in the extermination of the mosquito, suffered greatly from the inroads of mosquitoes which were bred by their neighbors.

This led to an attempt to formulate a law by which to get control of the whole problem, and be sure that, if there were six little towns

Dr. Hunt.

or villages in the county, instead of having two do the work and suffer from the lack of work on the part of the other four, some arrangement could be brought about whereby all six municipalities would do enough work to control it. That led to the passing of the present county law, by which, as Mr. Miller has stated, county commissions are established. Each commission is given funds in accordance with the law, rated by the taxable valuations of the county. The commission formulates its plans, states the amount of money which it will need for the season's work, and sets forth the need of the work to be done. This is forwarded to the Director of the State Experiment Station, and his "O. K." makes it mandatory on the board of freeholders to furnish the money on the request of the commission. The commission is appointed by the chief justice of the county. This relieves the situation and takes the question out of politics. The commission serves without pay, and, up to the present time, this system, as it is being tried in Essex County, has been found to be admirably adapted to the problem.

The Essex County Commission has met with much criticism, but in spite of that it has gone ahead with the work. The members have felt that they know the problem, know the mosquito and know how it breeds; they know how it can be controlled, and they have men willing to put in the time and do the work, and, under the new law, they have the money.

The law was framed so that the appropriation was not available until a certain proportion of the county had been flooded with mosquitoes which bred on the salt marshes, but, after that, the work was carried out as thoroughly as possible. As a result, both the local papers and the general sentiment of the communities were unanimous in the opinion that they had never known a season when mosquitoes were so few in Essex County as the one just past. Weather conditions may have been favorable for the Commission during 1912, but it is known that wherever mosquitoes have been found heretofore they were found to be breeding during the past summer, but there were enough men to control the situation. It seems almost useless to continue the discussion of the paper, because the speaker's work and that of Mr. Miller naturally have been on parallel lines; his plans are the speaker's plans on this subject, and the results are the same.

To reiterate: There is in Essex County, as Mr. Gray has stated, not only the salt marsh mosquito, but the house mosquito, the *Culex pipien,* which has proven a greater nuisance than either of the salt marsh varieties. The *Culex cantator* and *Culex solicitans* breed only in salt water. These mosquitoes have been found in Morristown, Summit, and Bernardsville, and also at points from 20 to 25 miles from the nearest salt marsh. It is apparent, therefore, that these

Dr. Hunt. varieties are migratory, and are capable of flying 25 miles. They are most prolific, and can be controlled only by ditching the salt marshes. The salt marshes on the Hudson County Meadows and on the Union County Meadows are being ditched thoroughly, and it is hoped and believed that the salt marsh mosquito can be eliminated from Essex County and from New Jersey, if this work is continued in its present form.

The *Culex pipien* is a small mosquito which breeds in fresh water, and the dirtier the water, the better it likes it. It breeds in brooks, in old tin cans, in pools, and in any place where there is water. It requires a house-to-house inspection for its control. It is a continuous nuisance, and to eliminate it each inspector must visit and revisit every house in his district. Essex County was divided into as many districts as seemed wise, and an inspector was placed over each one and made directly responsible to his chief, and the chief to the Commission.

The above-mentioned mosquitoes are practically the only three varieties which are found in Essex County. We know where they breed; we know how they breed; and we know how to beat them; and, if the money is still forthcoming in future years, we believe it to be inevitable that the mosquito must go.

All will acknowledge the fact that it would be a wonderful thing for New Jersey or for any community to rid itself of mosquitoes. One can hardly estimate just what it means in real estate values alone. It is within the experience of everybody who lives in New Jersey that men who have gone there to occupy property and to make it a permanent home, have suffered from this little pest, the *Culex pipieu,* and the salt marsh mosquitoes, and have gone away and refused to invest in property in that State. Therefore, in Essex County, it is considered a good investment to devote $75 000 toward the extermination of the mosquito and set it against the depreciation in real estate and the actual discomfort which comes from the pest.

The speaker is a practicing physician, and during 1912 has been somewhat interested in the amount of malaria in the county. He agrees with Mr. Gray in his statement that the mosquito problem is more of a nuisance than a health proposition; not that we do not have malaria in New Jersey, for we do have it, but it is a negligible quantity, and we would not feel justified in spending any such sum of money to get rid of what little malaria there is in the county. In doing away with this nuisance, however, we will destroy the *Anopheles* mosquito, which is the carrier of malaria.

The speaker is very glad that Mr. Miller has written this paper, and feels that everywhere we go we should preach this gospel. If intelligent men could be interested in the subject, another step in the fight would be won. It is hoped that each engineer will do all he can in his community to create an interest in this work and aid it in every possible way.

The mosquito does act as an intermediate host. The malarial organism goes through an entirely different stage in the mosquito from what it does in the body of man. This is also true in yellow fever, as far as is known. It cannot be traced in yellow fever, because the organism has not yet been determined, but in the case of malaria the organism has been traced in all its stages from man to mosquito and from mosquito back to man. Dr. Hunt.

ROBERT A. RUTHERFURD, ESQ.—In Connecticut, a number of people interested in the mosquito question, including the speaker, are trying to obtain some legislation, based on the effective New Jersey law, but modified to suit the different political and physical conditions. Mr. Rutherfurd.

The people are bothered to a great extent with salt marsh mosquitoes which can be eliminated by drainage of the salt marshes. One of the proposed bills places the entire direction and execution of the marsh-drainage part in the hands of the Director of the State Agricultural Experiment Station. Another bill deals with the breeding of fresh-water mosquitoes, and empowers boards of health to remove them like any other nuisance.

From all the preliminary missionary work that has been done, it looks as if favorable results would be obtained from the efforts which are being made at Hartford.

The proposed bill putting the marsh drainage work, etc., in the hands of the Director of the State Agricultural Experiment Station calls for an appropriation of $200 000, and gives power to investigate, inspect, and survey all suspected breeding marsh areas, and also gives him authority to hire engineers, etc., to survey and map such areas, and to estimate and inspect such work as required. The engineers should have a knowledge of mosquito breeding marshes.

Some private anti-mosquito work in several of the small towns has given very good results, particularly the ditching of salt marshes; and, of course, as Mr. Miller states, that has to be maintained from year to year, and kept in proper shape, or the mosquitoes go back and make worse breeding places than ever.

KENNETH ALLEN, M. AM. SOC. C. E.—Reference has been made to the breeding of mosquitoes in catch-basins. These basins may be very prolific sources of such pests in our cities, and very little has been done to prevent them from becoming breeding places. When in Washington, in September, 1912, the speaker was interested to see that active preventive measures were being taken to that end, under the direction of Asa E. Phillips, M. Am. Soc. C. E., Superintendent of the Sewer Department. The catch-basins, of which there are more than 5 000, are flushed regularly, once a fortnight, and are given a dose of about a cupful of oil during the breeding season. The cost of each application is about 6 cents per basin. The catch-basins are Mr. Allen.

Mr. Allen. cleaned once in 4 weeks throughout the year, the flushing being merely to remove the stagnant water.

Cesspools also afford favorable conditions for mosquito breeding, and it will be interesting to note what change may be observed in a city like Baltimore, where 100 000 cesspools are being abandoned on the introduction of sewerage, and where, in certain localities, this insect has been a serious nuisance.

The dissemination of disease by mosquitoes has been suspected for many years, although the conclusive proof is recent. Dr. Howard A. Kelly, of Baltimore, in a paper before the Johns Hopkins Medical Society, mentions some native writings of the Sixth Century, discovered by the Governor of Ceylon about ten years ago, in which insects are mentioned as carriers of disease, and certain kinds of mosquitoes which cause malaria are described.

In 1848 Dr. Josiah C. Nott, of Mobile, Ala., expressed the opinion that yellow fever was transmitted by insects, but it was not until 1886 that the disease was charged to the *Stegomyia fasciata* by a Scotchman then living in Havana—Dr. Carlos J. Finlay; and then some 14 years elapsed before this was demonstrated to be a fact by Dr. Walter Reed, U. S. A., and his colleagues. Referring to this, Dr. Kelly says:

"One thing must strike every reader of Reed's experimental work—that it is based on nothing which was not easily within the scope of scientific investigation at any time within the last 200 years. Their success rested upon inductive reasoning, accuracy of observation, and perseverance, and thus affords an added illustration of the old lesson that the simple solution is the product of the master mind."

Mr. Britton. W. E. Britton, Esq.* (by letter).—The writer has been much interested in this paper and realizes that the civil engineer has a great opportunity for effective work in mosquito elimination. Especially is this true in regard to construction work, where some of the very worst breeding places have been formed by carelessly intercepting the natural drainage in the building of railroads and highways across marsh lands.

The subject of mosquito control naturally falls into two subdivisions: (1), Salt marsh or migratory species; and (2), Malarial, rain-barrel, and other local species.

The control of the first species requires a large initial expenditure for drainage. As salt marsh mosquitoes migrate, often flying many miles, not only the area where they breed, but all the adjacent towns are affected. Local effort, therefore, is not satisfactory unless universal co-operation is established. This is difficult. It seems to be a wise provision, therefore, that such work be managed by the State, as is done in New Jersey. After the drains have been cut, a small

* State Entomologist, New Haven, Conn.

sum must be expended annually for maintenance, and the writer believes that this may be safely imposed on the towns, possibly with a provision that the State may do the work in case the towns fail. It must be done some way. Mr. Britton.

The second subdivision deals more particularly with a local nuisance, and more properly belongs to the province of the health officer. If the laws are insufficient, more authority should be given to the health officer and more funds provided in order to enable him to cope with the situation.

The control of the rain-barrel and malarial and other non-migratory species may best be effected by frequent inspections of premises, at the expense of the town or city, and by orders issued by local authorities, the cost of the treatment to be borne either by the property owner, or by the municipality or town. State supervision or interference is hardly necessary.

Whether or not county mosquito commissions are desirable will depend on conditions existing in the different States. In Connecticut, for instance, there are county public health associations composed of the town, city, and borough health officers and a few citizens prominently interested in matters of public health. Such organizations at their meetings discuss these problems and adopt general policies; and each individual health officer will solve his local difficulties with a stronger courage if he knows that the organization is back of his efforts.

In mosquito elimination, as in all other questions of great public import, however, a strong public sentiment must be obtained. Thinking people must be awakened. Often laws will help to create that sentiment, but they are of little value unless supported by it. A great educational campaign is needed, and every legitimate means should be adopted to an end which will improve the health, efficiency, comfort, and, ultimately, the financial condition of the people.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE SANITATION OF CONSTRUCTION CAMPS.

Discussion.*

By Messrs. Frank E. Winsor, W. E. Britton, F. W. Austin, and Harry G. Payrow.

Frank E. Winsor, M. Am. Soc. C. E.—This paper is a timely contribution on an important subject relative to which very little has appeared heretofore in the publications of the Society. Mr. Winsor.

The construction and sanitation of camps for the temporary housing of men have, generally speaking, received little scientific consideration among those responsible for their creation, and the author's suggestions are so materially in advance of common practice as to be considered by many, perhaps, revolutionary.

The proper sanitation of construction camps depends on a variety of considerations, and involves many diverse problems. The character of camp construction should depend on seasonal variations of climatic conditions, the length of time which the camp will be occupied, the nationality and intelligence of the occupants, the character of the country in which the camp is located—particularly whether in a populated or non-populated district—and other less important considerations.

The following discussion is mainly from the viewpoint of camps located in more or less populous districts, the occupancy of which extends over more than one working season. The methods of camp construction and sanitation described have been used successfully along large portions of the Catskill Aqueduct, a structure some 90 miles in length, extending from the Catskill Mountains to New York City, and now nearing completion. The work and methods outlined are both practical and reasonable, and are believed by the speaker to be adapta-

* Continued from January, 1913, *Proceedings*.

Mr. Winsor. ble, with suitable modifications in detail, to substantially all conditions. The subject may be conveniently treated under the following headings:

1.—Location of Camp.—Generally speaking, camps should be located on high, dry ground, preferably sandy; and, where located on a drainage area tributary to a domestic water supply, a site remote from water-courses should be selected.

2.—Water Supply.—A water supply of suitable quality and quantity for drinking, bathing, and washing must be provided, and, if the camp is to be used for any considerable period, the supply should be properly piped under pressure to the various buildings. A ground-water supply from deep wells, protected from surface pollution, is generally the most satisfactory, although under some conditions it may be allowable to use surface springs or surface streams, if their purity can be properly and continuously safeguarded. Dug wells, open at the top, are almost always polluted, and it is seldom safe to use water from streams and lakes, particularly in a territory at all populated, without proper purification. It is important, especially with sources other than ground-water, to make frequent analyses in order to secure a continuous suitable quality of the supply.

3.—Food Supply.—An intelligent selection and inspection of all food products, particularly milk and such articles as are used without cooking, is important.

4.—Waste Disposal.—The most efficient method of disposal of human excreta, including urine, is by incineration, and it has been found entirely practicable to collect and incinerate such wastes from camps of both large and small size. Various types of incinerators have been used, including several small, portable ones with seats and urinals attached; but, generally speaking, with camps of considerable population, with several years occupancy in prospect, it has been found more economical and practical to build special incinerating furnaces, properly housed, and to collect all the fecal matter in tight cans, which are placed under the seats in the closets and changed daily. Suitable closets for housing the cans, protected from flies by thorough screening, should be provided throughout the camp and along the line of the work. The speaker is of the opinion that, under all conditions, incineration is much to be preferred to the latrines suggested by the author.

With the fecal matter disposed of, the problem of the other camp wastes is materially simplified. Garbage should be collected and burned daily. Stable manure should be disposed of daily, preferably by incineration, although it may be allowable, in agricultural districts, to permit its use on land, if it is hauled a sufficient distance from the camp to make sure that any flies which might breed in it cannot reach the camp area. Liquid wastes from the camp, other than those previously mentioned, may be taken care of in a variety of ways,

depending on local conditions. If the camp is located in a drainage area tributary to a public water supply, particularly if the drainage from the camp enters such water supply near an intake, extreme precautions are necessary. In the case of camps located on the drainage area of Croton Lake (the main source of supply of that portion of New York City which includes the Boroughs of Manhattan and the Bronx), the following precautions were taken: The camp was surrounded by intercepting ditches, and the rainfall on the area inside these ditches was collected in a settling basin of sufficient capacity to hold the rainfall from substantially all storms. From this settling basin the water was run through sand filters. These sand filters also filtered the wash-water from the camp, this water being collected by a proper sewerage system. The effluent from these filters was afterward dosed with chlorinated lime, so that a substantially sterile effluent was finally obtained—generally much better than the natural flow of the stream into which it emptied. In case of camps located in polluted drainage areas not used for domestic water supply, the omission of filtration for surface run-off from the camp and the substitution of septic tanks or broad irrigation for wash-water, etc., may be permitted. Mr. Winsor.

5.—*Character of Construction of Buildings.*—Ample provision for air space and ventilation should be made in the buildings, and a minimum allowance of 400 cu. ft. of air space per occupant is believed by the speaker to be none too much. Proper heating in the winter season and suitable lighting should be insisted on. The mess-house should be detached from the other buildings. Hospitals of proper size and suitable equipment should be provided, and a building which may be used as an isolation hospital should be in readiness in case of contagious diseases.

6.—*Medication.*—A physical and medical examination of applicants for employment is very desirable, and may avoid much subsequent trouble. The continuous attendance of a properly qualified physician, to take charge of the sanitation of the camp and the health of the occupants, is very important, particularly in camps housing a considerable number of men.

7.—*Protection from Flies.*—One of the most important problems in sanitation is to prevent, as far as possible, the breeding of flies, which are believed by the speaker to be one of the greatest pests in existence. The daily collection and disposal of all camp wastes, in most of which flies may breed, should be rigidly insisted on, and all quarters for the men, and, in fact, all buildings in the camp, should be properly screened.

8.—*Fences and Lighting.*—As extreme precautions, high fences of a man-proof type entirely surrounding the camp, with one or more openings which can, when desirable, be placed under proper guard,

Mr. Winsor. and illumination of the camp area at night, have been found very effective as aids in securing compliance with sanitary rules.

9.—Police.—An efficient police force, while not essential, is a valuable adjunct in connection with securing the best results.

There is no doubt in the speaker's mind as to the economy of proper camp sanitation. It has been demonstrated as both possible and practicable to eliminate substantially the scourge of typhoid fever and other contagious diseases, and to secure sanitary conditions in temporary camps in advance of those found in most established communities. There is, no doubt, a great increase in the individual efficiency of men employed under proper sanitary conditions, and a large gain also due to non-interruption of the work on account of sickness among the force. The restriction as to time is very important in nearly all construction work, and expenditures for adequate camp sanitation, which should never exceed a small percentage of the cost of the work, will almost invariably be returned several-fold in economy resulting from increased efficiency of the force, earlier completion of the work, and other indirect and less tangible benefits. The safeguarding of the health of the surrounding country is also a most important factor in work located in populous districts.

The efficient enforcement of sanitary rules is a very difficult matter with the class of labor ordinarily employed on construction work, and the immediate discharge of employees who are discovered violating these rules has been found to be the most effective means of securing their enforcement. A continuous campaign of education is also of great value as an aid to securing the best results.*

Mr. Britton. W. E. BRITTON, ESQ.† (by letter).—The importance of camp sanitation has long been overlooked by both engineers and contractors, doubtless for the reasons given in Mr. Gray's admirable paper. Recent discoveries, which show how many communicable diseases are transmitted by insects, render strict rules and regulations imperative, and sanitary precautions and appliances essential. As regards the laborers, ignorance is their chief excuse, but this is not so with the engineer, who should also be a sanitarian, for prevention is better than cure.

The construction of the Panama Canal was primarily a problem of sanitation, and could not have been solved from other standpoints. The average contractor is probably more easily moved by a desire to further the progress and profits of his work than by humanitarian and philanthropic motives. For this reason, an appeal for sanitation

* A very complete and able paper by Andrew J. Provost, Jr., Sanitary Expert, entitled "Sanitary Problems of the Board of Water Supply," outlining the methods and results of sanitation along the line of the Catskill Water-works, the new water supply for New York City, was presented before the Municipal Engineers of the City of New York, and is published in the *Proceedings* of that Society for 1911

† State Entomologist, New Haven, Conn.

on the grounds of increased efficiency of his men and to prevent economic loss seems to be wise. A few damage suits for criminal negligence in extreme cases might have a salutary effect. Of course, there are many construction, lumber, and mining camps located far from other human habitations, and, in such cases, a neglect of sanitation affects only those connected with the camps. On the other hand, there are construction camps located on the outskirts of many cities and villages, and here a neglect of sanitary measures imperils the lives and health of thousands, though, in most cases, such camps would be within the jurisdiction of the health officer. Mr. Britton.

After the resident physician, the engineer is probably the most important officer concerned with the establishment of construction camps, and it is to him that we must look in the future for a large degree of improvement in camp sanitation. If he has a knowledge of the dangers and will assume the responsibility of avoiding and remedying them, we are making real progress. It is gratifying that papers of this nature are presented before the Society.

F. W. AUSTIN, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Gray is to be congratulated for the manner in which he has presented this subject. The proper sanitation of camps should appeal to many construction engineers and contractors, and it is to be hoped that better sanitary conditions may be obtained. Any method or system, however, to appeal to the contractor, should consist of appliances which are easily and economically obtained. For example, Mr. Gray stipulates that copper or bronze screens should be used to protect the various openings, such as windows, doors, etc. The contractor is usually situated where copper or bronze cloth is not readily or economically obtained, and it is not quite clear to the writer why common iron screens will not answer the purpose quite as well, as they are for temporary use and all temporary buildings are abandoned by the contractor or sold for a nominal sum when the work is completed. Mr. Austin.

Perhaps the greatest obstacle to be overcome will be the lack of discipline. In the average contractor's camp, a large percentage of the laborers employed remain for a short time only, in some camps from 5 to 15 days is a fair average, and many of them are not enthusiastic about shaving and taking baths. They are also not much given to worrying about disciplinary measures, such as being suspended from the payroll or discharged from service. It is evident, however, that much improvement can be made. It will be necessary to arouse more interest on the part of contractors, for, as a rule, they do not take much interest in the sanitary condition of their camps.

If the company letting the contract would insert a clause requiring the contractor to keep his camp in a proper sanitary condition, the

Mr. Austin. engineer in charge would then be in a position to enforce the necessary rules and regulations to bring about the desired result. This would not only improve the efficiency of the service, but would tend to create a favorable impression among the people living in the vicinity of the camp.

Mr. Payrow. HARRY G. PAYROW, JUN. AM. SOC. C. E. (by letter).—Until recent years, the sanitation of construction camps received little attention by health authorities, due partly to the fact that most camps are remote from populated districts and that the authorities are indifferent. On the other hand, engineers and contractors have not manifested as great an interest as they should, for, as the author states, camp conditions, to a large extent, govern the efficiency of the labor.

The following is a brief description of the methods in actual practice in a construction camp with which the writer was connected. These methods are believed to be simple, of low maintenance cost, and efficacious.

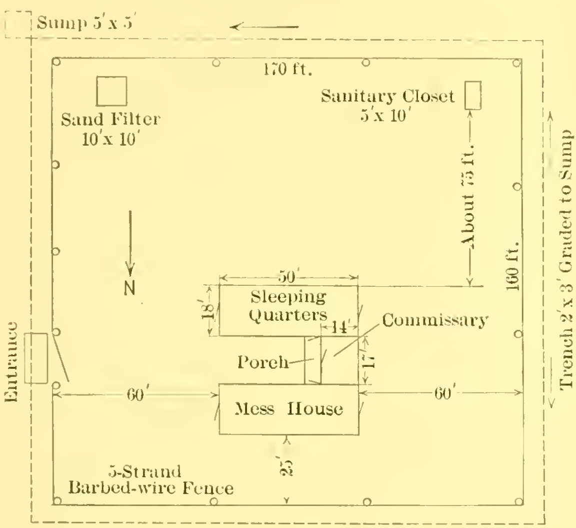
This camp was on the water-shed of a reservoir used as a city water supply, and at that time an additional $1 000 000 dam was under construction. For convenience, the camp was near the work and on slightly sloping ground. An enclosure was formed, with a man-proof fence of 5 strands of barbed wire, and only one gate was provided. Outside of this fence a 2 by 3-ft. trench was excavated all around the enclosure to catch the storm-water, thus preventing it from reaching the reservoir. The trench was graded to carry the water to a sump where it filtered away.

The sleeping quarters consisted of a well-built, one-story wooden building, covered with weather paper. Instead of the old style bunks, there were comfortable iron beds, a few being on a balcony along one side of the room. A large coal stove was used for heating. The mess-room was a separate, one-story building, and contained long tables with benches and two or three cook-stoves. Connecting the two buildings there was a commissary, where supplies of good quality were sold at reasonable prices. All windows and doors were effectively screened.

The sanitary closets were within the camp enclosure, so that the men would not be tempted to go elsewhere. These were ordinary closets, except that galvanized-iron ash-barrels were used as receptacles. These barrels were taken away regularly every day to a farm situated far from the water-shed, and there emptied, washed, sprinkled with lime, and returned the next day. All garbage was also disposed of in this manner, barrels being placed in convenient places for the disposal of waste matter. At the height of fly-time all sanitaries were sprayed with a formaldehyde solution, which prevented to a large extent the accumulation of flies. For washing purposes, a small sand filter was constructed within the enclosure, and all wash-water was thrown on the filter.

There was a caretaker for the camp, and it was his duty to see that beds were aired daily, floors swept, disinfectant sprinkled about (especially in the sanitary closets), and that the men did not throw refuse carelessly about the enclosure. The patrolman of the water-shed, employed by the city, visited the camp at least once a day and forcibly reminded the caretaker of his responsibility. Mr. Payrow.

The sanitaries on the work were provided with ash-cans which were collected daily and cleaned, as before stated. Any man on the work who was so forgetful as to fail to use the sanitary closet was either laid off or discharged, according to the seriousness of the offense.



SUGGESTED PLAN
OF A
CONTRACTOR'S CAMP FOR 30 TO 40 MEN

Fig. 3.

Fig. 3, a plan of a contractor's camp, is submitted merely as a suggestion for a layout. Additional sleeping quarters can easily be provided if a larger number of men are to be employed. It is quite essential that there be plenty of yard room surrounding the building, for this tends to keep the men, while idle, within the enclosure.

After the men understood what was required of them, they gladly conformed to the regulations and very few breaches occurred. They enjoyed the camp, very few of them left the contractor, and there was no trouble in getting men to come on the work. This was in marked contrast to another camp, of the old bunk variety, on the same work. This was conducted by another contractor, and the men were continually disgruntled and constantly leaving.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

IRRIGATION AND RIVER CONTROL IN THE COLORADO RIVER DELTA.

Discussion.*

By Messrs. L. J. Le Conte and Morris Knowles.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This interesting paper furnishes valuable information relating to the troubles of the irrigation engineer and the irrigators. A record of the memorable struggles with flood-waters in the Colorado Delta in recent years will always be a source of valuable information. Mr. Le Conte.

The rich fertilizing qualities of the silty waters of the Colorado River are well known. The important point to keep in mind is the fact that the rich and desirable sediment always comes down with the first rising river waters, and the undesirable barren sandy silt during high water and the falling river stages. It follows, therefore, that the desirable silt is to be had largely from the rising river waters, and that suitable management and wise control of the scouring sluices and regulating gates at each end of the Laguna Weir are of the greatest importance in the maintenance of the entire canal system. In order to reduce silting troubles to a minimum, the mean velocity of flow throughout the entire canal system should be as nearly uniform as practicable.

For this purpose the formula given by Mr. R. G. Kennedy,† is very useful, namely:

$$\text{Velocity} = 0.84\, d^{0.64}.$$

*This discussion (of the paper by H. T. Cory, M. Am. Soc. C. E., published in November, 1912, *Proceedings*, and presented at the meeting of January 8th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† *Minutes of Proceedings*, Inst. C. E., Vol. CXIX, p. 281.

Mr. Le Conte. hence, comparing one channel flow with another, we have practically the ratio:

$$v : v_1 : : \sqrt[3]{d^2} : \sqrt[3]{d_1^{\,2}}$$

but, after all, the main feature to keep constantly in mind is that the mean velocity varies as $\sqrt{R\ S}$, and that S is the slope of the surface of the water and not the slope of the bed of the canal. With the best of management, however, there will always be some silting up, but the silt ought to be largely of valuable character, and when dredged and deposited on the bank adjoining the county roads the farmers will eagerly cart it away for fertilizing purposes. The clean, barren, sandy silts which are so undesirable, should not be allowed to enter the head-works of the canal to any large extent. The cost of the Laguna Weir, $434 per lin. ft., when compared with the cost of the Kistna Anicut, in India, $104 per lin. ft., seems to be unreasonable, but such comparisons are always odious especially when all circumstances are not known. The annual revenues from the Kistna system amount to $1 500-000, and if the Laguna Weir system approximates this figure, in the course of events, all will be satisfied.

The author's remarks about a "rating curve" are commendable. No official record is so absolutely unreliable as the conventional rating curve for any given gauging station. It is worse than no record whatever, inasmuch as it is a worthless official record which will have to be officially disproved in Court before one can get authority to proceed. Experience everywhere on sedimentary rivers shows that all the cross-sections of a rising river are enormously increased in area by the scouring out effect of the down-rushing flood-waters ladened only with lighter sediment. The heavy sandy sediments gradually crawl down river much more slowly than the rushing flood-waters, hence they do not show until the river begins to fall. It follows that all the cross-sections of a falling river diminish rapidly in area and return gradually to the usual low-water cross-section. These being the facts on every flood, how is it possible for any sane man to get up a rating curve which has any practical value whatever? This is particularly true in the case of a gauging station on the main river just above a large tributary stream.

When the tributary stream is in high flood, the water in the main river is not only high, but the current is often running up stream, which potent fact alone destroys completely the value of the rating curve.

The author's proposed plan of building a wide rock fill dam, without any concrete cross-walls whatever, seems to be plausible, in view of the valuable experience in closing the big breaks in the Colorado River banks leading down to the Salton Sea.

These rock fill dams have now stood the ravages of severe floods for the past 6 or 7 years, and are still intact and water-tight, to all practical

purposes. The main object of the concrete cross-walls in the Indian dams is to shut off leakage through the broken stone fill and hold the stones together. As the low-water flow of the Colorado River is small (6 000 cu. ft. per sec.), it becomes very important to hold all the water possible. If a successful, wide, water-tight dam can be made, as described, without them, all well and good. The writer is of the opinion that it is well worth trying, from a financial point of view, particularly at such a site as the Laguna Weir, where the slope per mile is only 1.2 ft. Mr. Le Conte.

In India, the anicuts are generally built where the slope per mile is 3.5 ft. and the river bed is pure sand of unlimited depth.

Mr. Knowles. MORRIS KNOWLES, M. AM. SOC. C. E. (by letter).—This excellent and exhaustive paper is a most fascinating story of the ever-changing Colorado. The reader can almost picture himself on its banks, witnessing the frequent breaks, overlooking the vast overflows, and taking part in the repeated attempts at closure. The Engineering Profession can indeed feel gratified that it has members who will patiently chronicle events, analyze situations, and faithfully tell of failures.

Much is related of irrigation needs, local causes and effects of floods, notable works built and projected for the former, the great emergency construction to overcome the latter, and studies and plans for future protection and control. Little has been said, however, of the possibilities of prevention, of great works that might be built to hold back the flood flows and let out at other times the increased quantities that would make for greater irrigation possibilities, increased navigation stages, if otherwise possible, more water for power, etc.

Within a short time, and brought to a focus by the work of the Pittsburgh Flood Commission, the country has realized that we can no longer be indifferent to the vast and universal country-wide problems of regulating the flow of our streams. Without going into details of the many correlated benefits to be obtained from storage of flood flows, the devastations of the Ohio, Missouri, and Mississippi Valleys, those of the Southern Appalachian regions, as well as those of the Pacific Coast and great Colorado, so vividly portrayed in this paper, have awakened people, all over this land, to the fact that here is an important problem to solve, in which the factors have some relation, wherever we go, and from the solving of which, much other good will come.

That this is also true of the Colorado Basin is evident from the author's remarks on pages 1352 and 1353* and from Table 2. It would be of still further interest, therefore, to know whether the storage, of approximately 10 000 000 acre-ft. exhausts the possibilities on the Upper Colorado, and what effect such impounding at several selected

* *Proceedings*, Am. Soc. C. E., for November, 1912.

Mr. Knowles. points would have in decreasing the flood discharge and increasing the low-water flow. It is possible that it would not apparently diminish the interest and intensity of the problem of protection below Yuma. on account of the peculiar local characteristics, yet it is probable that the cost of such works would be less and safe results more certain.

If data are available to show the effects of such storage of flood peaks, as is illustrated by the Pittsburgh report, it is hoped that the author will present them in his closure. If not now obtained, perhaps this discussion may call attention to the need, and emphasize the fact that here is another place and a further reason for the Nation, by its Central Government, to take hold of its water problems as a great national question, and to study, plan, and treat each group of streams as a unit from source to mouth.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

CHARACTERISTICS OF CUP AND SCREW CURRENT METERS PERFORMANCE OF THESE METERS IN TAIL-RACES AND LARGE MOUNTAIN STREAMS STATISTICAL SYNTHESIS OF DISCHARGE CURVES.

Discussion.*

By Messrs. W. G. Price, E. E. Haskell, Charles H. Miller, and John C. Hoyt.

W. G. Price, M. Am. Soc. C. E. (by letter).—This paper gives the results obtained from a very large number of observations, and should be of much value. Before accepting these results as perfectly reliable, the writer would be pleased to know if the meters, in both the rating and stream tests, were suspended in precisely the same way and by supports of the same size? Mr. Price.

Were weights used in both cases; and were they always the same distance from the wheel?

Was the skiff so far from the wheel that it did not push the water ahead of itself a trifle and thus reduce the rate of revolution of the wheel? In some cases, a steamer ascending the Upper Mississippi will check slightly the flow of the water some hundreds of feet ahead of itself.

The writer has had an extensive experience with current meters during a period of seventeen years, beginning with 1879, during which time he measured the flow in the Mississippi, Missouri, and Ohio Rivers, and their large and small tributaries, many times, and in many places. The following propositions are the result of his experience:

* This discussion (of the paper by B. F. Groat, Assoc. M. Am. Soc. C. E., published in December, 1912, *Proceedings*, and presented at the meeting of February 5th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Price.

The thread of a perturbed current, of course, is longer than that of a steady current. A meter of the cup type, without a vane, measures the thread of the horizontal current perturbations, but when provided with a vane, it measures a trifle more than the thread of the horizontal perturbed current. This is due to the effect on the vane of the perturbed current which swings the wheel transversely to the current. If a cup meter with no vane is fixed on a vertical rod, the current perturbations moving up and down in the vertical plane will decrease the revolutions of the wheel and thus offset the increase due to the horizontal perturbations. This action is due to the fact that the top and bottom faces of the meter wheel present a propeller-shaped surface to the water, and when vertical currents strike the wheel they tend to turn it in the opposite direction from that produced by the water flowing against the cups. For this reason the vane should not be used in streams shallow enough to permit of it being omitted, if there are perturbed currents. Larger streams, when the gauging station is carefully selected, are not perturbed to such an extent as to cause a serious error in meter measurements.

A meter of the propeller type will measure less than the thread of the perturbed current, because of the inability of the vane to keep the face of the wheel at all times precisely normal to the current. A current striking a propeller wheel sidewise does not increase its revolutions, as is the case with the cup meter. Each type of meter, in clear, straight, running water, will give the same accurate results, provided it has been accurately rated and is supported in the stream in precisely the same way as when rated, or by a method which does not produce an increase or decrease of current at the meter wheel.

When a weight is attached to the supporting rod under the meter, it reduces the discharge area and increases the velocity of the water in its vicinity, and thus increases the rate of revolution of the meter wheel. If the weight is placed 6 in. below the meter, the wheel, in each type, will run much faster than when the weight is 12 in. below the meter. A difference in the location of the weight, or the use of the weight in one case and not in the other, will produce all the difference, noted by the author for the Price meter, between ratings and stream measurements.

The propeller type of meter is often made to run more slowly between rating and stream measurements by a slight quantity of grit blowing into the bearings, or being carried in by the water. This type of meter is often made to run slow by a particle of dead grass or leaf (found moving in most streams) catching on the propeller. The propeller type of meter does not free itself from such débris, as does the cup meter.

When meters are rated in still water, it is necessary to wait 10 min. or more after each passage of the meter for the water to become still again. If the wind is blowing against the surface of the water, variable and sometimes rotary currents will be produced, which will make the rating inaccurate. The surface currents will run in one direction and the under current in the opposite direction. For this reason, the meter should be rated at a depth of more than $2\frac{1}{2}$ ft., and in water more than 5 ft. deep. Mr. Price.

A rating taken in running water is likely to be the most accurate, because the running water carries away the disturbing currents produced by the movement of the meter.

When a cup meter is accurately rated in still water and in ordinarily steady running water, the two ratings will be the same within a small fraction of 1%, and the rating curve can be drawn through all the plotted points obtained by the rating, so that no point will vary in revolutions as much as 1% from the curve.

The writer has used a Morse paper tape register and electric device to start and stop the register, or mark the paper tape, at the instant of passing the cross-wires suspended over the 200-ft. base, so that the fractions of revolutions of the cup meter wheel were determined at the beginning and end of the run; and such a rating in still water, with almost no wind, located all the plotted points within a very small fraction of 1% of the number of revolutions from the rating curve as drawn.

The ratings of both the Haskell and Price meters given on Plates CXXXIV and CXXXVI* are inaccurate, as the plotted points indicating the number of revolutions vary entirely too much. The errors in the Price ratings are apparently much greater than in the Haskell ratings.

Mr. Groat has conducted a series of observations in one tail-race where the currents were quite divergent, and the writer submits that the result of his experiment, treating as it does of one special condition, cannot be taken, either directly or by inference, to apply to general conditions of measurement of the flow of streams.

Under such peculiar conditions, where there are cross-currents, as in the Massena experiment, it is possible that the propeller type, owing to its peculiar construction, is less affected by such currents than the cup type; but if the propeller type were better for the measurement of streams where accurate measurements are impossible, owing to cross-currents, this fact would not indicate that such meters are better for general use.

A perfect still-water rating, of course, is what is required when the instrument is to be used in the measurement of straight, flowing

* *Proceedings*, Am. Soc. C. E., for December, 1912.

Mr. Price. streams. The usual rating in running water is made only to secure the same result as would be obtained by a perfect still-water rating. The small variations of the current from a straight line, as seen at the gauging section in most streams, makes only a negligible error in meter measurements.

A meter measures approximately the length of the curved thread of the current; therefore, if the thread is very divergent, inaccurate results will be obtained with any meter.

A gauging section having a perturbed current which would cause an error of 6% would give evidence to the eye by the boiling of the water, and would make the experienced observer hunt for a better section. However, better sections cannot possibly always be found.

A gauging section which has an up-stream current should never be used, and one which has very divergent currents cannot give accurate results with any method of measurement.

The first meter used by the writer had a cup wheel which revolved in a horizontal plane. It was designed by Gen. Ellis, and had been used by him in the Connecticut River. This meter did good work in water which was absolutely free from grit. Floating organic matter, such as leaves and grass, did not often affect it, as the turning of the wheel caused such débris to free itself. When the attempt was made to use it in the muddy water of the rising Ohio in 1882, it gave very inaccurate results, owing to grit getting into the bearings, which made its rate variable. The writer was then furnished with two very finely constructed meters, designed by Clemens Herschel, M. Am. Soc. C. E., and made by Buff and Berger especially for this work. Each was of the propeller type, with horizontal shaft, having agate bearings, and each was provided with a vane to hold the meter in line with the current in both the vertical and horizontal planes. They were as perfect instruments of the propeller type as the writer has seen.

These meters were tested by rating them in clear water, and gave good results. They were then rated in a bayou of roily water, an overflow from the Ohio, and the results were very inaccurate, owing to grit getting into the bearings, and leaves and other débris catching on the wheel and continuing to revolve with it. They were tested in the turbid Ohio River with no better results.

It was absolutely necessary to have a meter which would do accurate work, if the great Ohio flood of 1882, which was then on its way, was to be accurately measured. In designing and constructing what is called the Price meter, on this occasion, the cup type of wheel was adopted, for three reasons: The friction of the bearings is so small that this wheel can measure sufficiently low velocities, which is not the case with the propeller type; the revolving wheel will clear itself of débris; and, by using a vertical shaft, with inverted cup bearings filled with air and oil which will stay in by capillary attraction,

no water with its load of grit can enter. The friction of the bearings thus becomes a constant quantity. The well-known and successful operation of these meters is due to these three features, which the writer believes cannot be secured by any other design. Mr. Price.

The propeller type of meter, designed by Mr. Herschel, was in service before the Price meter, but has had quite a limited use, while several thousand Price meters have gone into service. This test of time, under all kinds of conditions, indicates that the cup meter is the most efficient for general purposes.

The writer believes that with an accurately rated cup meter of the type having no vane—which is much the better design for shallow, perturbed streams—more accurate results could have been secured at Massena.

E. E. Haskell, M. Am. Soc. C. E. (by letter).—The writer has been greatly interested in the results set forth in Mr. Groat's paper, for he has never had occasion to use current meters under just such conditions. Mr. Haskell.

The current meter was designed originally to measure velocities in open channels. It has developed into two distinct types: those having screw wheels and those having cup wheels. The first current meter, "Woltman's Mill", more than 100 years old, was of the screw wheel type. This meter has undergone various changes in form and in the mounting of its wheel, and, on the whole, has proved to be very accurate and reliable. The origin of the cup wheel meter is not as definite. It has undergone various changes, and the cups have been of various forms, namely, cones, hemispheres, and combinations of cones and hemispheres. Although a great deal of good work has been done with meters of this type, they have not shown themselves to be anywhere near as reliable as those of the screw wheel type. In the writer's judgment, this is due to the fact that they do not always present the same area and form of wheel to the current pressure. The tilting of the meter, even through small angles, changes these, and consequently changes the rating curve or equation. In current meters of the screw wheel type, such tilting through small angles makes but a slight change in the area and form of wheel presented to the current pressure, and, accordingly, they follow much more closely their rating curve or equation.

In the last paragraph on page 1678,* the author states, as follows:

"A conclusion to be drawn from these experiences is that a tail, or rudder, is a useless appendage to a meter used in stream gauging."

The writer's experience will not allow him to accept this conclusion. In a large river, the difficulty of holding the axis of the meter perpendicular to the cross-section becomes so great that it is out of the

* *Proceedings*, Am. Soc. C. E., for December, 1912.

Mr. Haskell. question. In large streams it is necessary to measure both factors, velocity and direction, and reduce to obtain the velocity normal to the cross-section. In small streams or tail-races it is possible to hold the meter rigidly in place; but, when thus held, if the direction of the current makes an angle with the horizontal, that is, strikes the wheel of the cup meter either on the top or bottom of the cups, it will over-register as readily as if running in "eddies" or "boils". The amount of this over-registration depends on the angle which the direction of the current makes with the horizontal, and can only be determined by some check method.

It is not at all clear to the writer why a still-water rating of a current meter should not give an entirely satisfactory result. In all cases where the elements of the current in the open channel are reasonably parallel, the meter is working under almost the same conditions as when drawn through still water. In one case the meter is free and the water moving, and in the other the meter is free and moving, and the water still.

The following test of a direction current meter was made after it had been rated in still water. It was made in a large tidal channel, free from "eddies" or "boils", where the elements of the current were parallel. Each comparison was made by running the meter for 10 min., and in this time running ten large cylindrical can-floats, alternating on each side of the meter. The determined velocities are in feet per second, as follows:

Float.....	1.04	1.29	1.55	1.94	2.56	2.65	2.76	2.80	3.08
Meter....	1.05	1.28	1.55	1.96	2.59	2.64	2.74	2.82	3.11

The greatest uncertainty in the results obtained with cup wheel meters is caused by using such meters from an unstable support, like a boat, particularly if the weather conditions are such as to give the boat any rocking or pitching motion.

The author is deserving of much credit for his work and his paper. Well planned and well executed work cannot help but make for engineering progress.

Mr. Miller. CHARLES H. MILLER, M. AM. SOC. C. E. (by letter).—Although the writer has made scarcely any current meter observations during the past twelve years, and hence has not kept up fully with the progress in such work, he is much pleased at the opportunity to read such a clear presentation of a series of observations, evidently made with much painstaking care; and is gratified to note such a complete verification, stated, however, in a much clearer manner, of a positive difference between a cup meter and a screw meter when running in perturbed water; which difference the writer noted just a short time before leaving the Government service, and has never had opportunity to investigate further.

Mr. Miller.

Although the difference was then cited as being due to instability of support, in other words, the rocking of the skiff, this rocking was done for the purpose, not only of causing conditions actually to be found in the river on account of "boils" and "eddies", but to show the effects that might be expected should the skiff or support be moved otherwise than in a straight and even course, while taking a rating in still water. That the writer was familiar with this source of error a number of years ago may be noted by referring to a letter* written by him, in which, in speaking about the rating of meters in skiffs, it is stated that "a rating should never be taken in a skiff which is being pulled with the oars," and that "the observers should be cautioned about keeping the same positions and not swinging their bodies from one side to the other while the observation is being made." This same passage was noted by the late William Starling, M. Am. Soc. C. E., and referred to in his paper entitled "The Discharge of the Mississippi River."†

Mr. Starling's paper treats in general of the many difficulties attending the taking of discharge observations on the Lower Mississippi River, and explains the reasons for some of the apparently discrepant results published in those days, he having had access, at the time, to the large mass of data gathered by the writer and others then engaged in wrestling with the problem of determining the extent of, or in eliminating, errors. It must be remembered that the possibility of error is very great when measuring the discharge of a river in which a favorable cross-section has a width of approximately ½ mile, with a maximum depth of 105 ft. at times, where velocities of 9 ft. per sec. or greater are found, and a total discharge of 1 750 000 cu. ft. per sec. may be obtained. Under such conditions, it is impossible to anchor the boat or equipment, and it must be held on one spot, as nearly as possible, by means of the rudder and the propelling power, counterbalancing the current and wind forces. Careful observations were made to determine how much movement actually took place. In fact, a large portion of the time in the field was devoted to checking the accuracy of the observations. It may be noted that the meters then used were of the cup variety. Some checks, however, were made by a system of double floats.

As noting some of the improvements that were being made from time to time, attention is called to the fact that, at a time as recent as 1891, the meter rating was considered as following a straight line, and the proper straight-line equation was calculated by the method of least squares, often, however, not until some time after the field observations had been made. About the date just mentioned it became customary to plot the rating observations and draw the curve

* *Engineering News*, January 24th, 1895, p. 58.

† *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 347.

Mr. Miller. conforming as nearly as possible thereto, from which a rating table was taken directly. Other tables were made to cover all necessary computations in the discharge work, and each station observation was calculated while the boat or steamer was moving to the next station, so that the total discharge was known a few minutes after the last velocity observation had been made. For instance, during the morning of May 4th, 1892, at Arkansas City, Ark., the discharge was computed at 1 733 676 sec-ft. as compared with 1 532 875 sec-ft. for the day before. Another observation, made during the afternoon, gave 1 741 526 sec-ft., showing that some marked change had taken place during that time. On the following day it had dropped back to 1 589 172 sec-ft., or to about the general average at that time. Under the old methods, the observations were not computed until some time, often weeks, after the field observations had been made.

As illustrating the fact that every method that could be thought of at the time was used to eliminate errors, it will suffice to say that, in the case of a large and persistent difference in the discharge, as found at two stations 90 miles apart, new meters were purchased, later they were interchanged, frequent ratings were made, and finally the observers exchanged places with each other; all this was done without finding a satisfactory reason for the difference. This was in 1892. During the high water of 1893 these same stations were again occupied, and further efforts were put forth to get accurate results. Mr. Starling, in the paper previously cited, has this to say regarding this work:

"In 1893, the discharges at Arkansas City and Wilson's Point agree fairly well; and when checked by the results of the flanking observations, they present a very coherent mass of data, affording the most trustworthy measurement of the extreme high-water discharge ever obtained on the Mississippi."

Too much stress cannot be placed on the fact that it is just as important to avoid errors in rating meters as in taking the actual discharge observations; and the writer is strongly of the opinion, based on long experience in the field, that by far the larger part of the error is due to the fact that ratings are usually made under physical conditions which are widely variant from those under which the discharge observations are made. For instance, if the observations are to be made from a steamer, then the ratings should be made from the same or a similar steamer, and in dead water, if possible, although they can be made accurately in moving water, provided the movement is smooth, generally uniform, and slow enough to permit the taking of slow velocities in the down-stream direction. Care must be exercised in keeping the meter below the influence of the "drag" occasioned by the steamer hull moving through the water.

Necessarily, the official reports, on account of limited space, as a rule, can contain but a brief explanation of methods and results; and,

naturally, these reports incline toward the better showings, failing in general to recognize the fact that the publication of actual failures will often hasten the time of the final solution of the problem. Mr. Miller.

It may be of interest to know that the first forms of meters were of the wind-mill, screw, or propeller type; that is, the vanes turned, in a plane which was perpendicular to the direction of the current. These were used on South American rivers in 1871.

To eliminate the tail or rudder appears to be a move in the right direction, but a vertical rod could hardly be used with success in gauging a stream as large as the Mississippi, and it would no doubt be possible to work out a design which could be mounted concentrically on the cable which is now used to lower the meter, with its attendant lead weight, to depths of 60 ft. or more, affixing the tail or rudder to the lead weight, and placing the latter so far below the meter that it would not influence its running.

John C. Hoyt, M. Am. Soc. C. E.—Although more than fifty devices for measuring the velocity of water have been patented, the only meters which have had general use are the Price, Haskell, Fteley, and Ellis, or modifications of these. Mr. Hoyt.

Each of the various meters has first been devised to meet the requirement of some special condition, and may or may not be applicable to other conditions. Therefore, a current meter should never be used for other conditions than those for which it is designed or has been found to be applicable.

In 1888 the United States Geological Survey began gauging streams of all sizes and in all sections of the country. These streams presented an infinite variety of combinations of range in depth, width, and velocity, were often remote from railroad or other regular transportation facilities, and were consequently difficult and expensive to gauge. No adequate instruments or methods had been developed for work of this varied nature. Furthermore, elaborate equipment and methods were out of the question on account of the limited funds. It was necessary to devise or adapt a current meter which could be readily carried in the field and operated by one man, either from a bridge, boat, cable and car, or by wading.

To fulfill these requirements it was essential: (*a*) that the meter should be simple and light in construction, with no delicate parts which easily get out of order; (*b*) that it should be simple in operation, including its preparation for use under any conditions, and its dismantling, cleaning, and boxing after use; (*c*) that it should offer a small area of resistance to the action of the water; (*d*) that it should have a simple and effective device for indicating the number of revolutions of the wheel; and (*e*) that it should be adaptable for use under all conditions.

After experimenting with various types, the engineers of the Survey

Mr. Hoyt. developed a meter combining certain essential features of the Price acoustic and the large Price electric meter. This is known as the small Price electric meter, and is now in general use in the Survey work. Modifications in its construction have been made from time to time, until now it represents the ideas of many engineers based on results of more than twenty years of experience in stream gauging.

The limitation of the Price meter pointed out in this paper, namely, that it may be affected by cross-currents, has always been recognized by its designers and users. Therefore, special care has been taken to restrict its use to work where the conditions are such that errors from this cause will not affect the final estimates of discharge. These requisite conditions are: (*a*) a fairly smooth bed; (*b*) a measurable and steady velocity of current; and (*c*) a stationary stage during the measurement. The velocity of the current should vary gradually throughout the section, which should show no marked eddies, cross-currents, or boils, and its mean should not be less than 0.5 ft. per sec. at low stages. After visiting hundreds of gauging stations, on streams of varied character, in all sections of the United States, the speaker is convinced that only the exceptional station is not or cannot be located in accordance with these requirements. This statement is made in order to correct any impression which may be drawn from Mr. Groat's paper, that the Geological Survey records, based on measurements with the small Price meter, generally involve large errors. More than two hundred engineers have been engaged in the stream gauging work of the Survey for various periods during the last twenty years, and, as far as known, no valid criticism of the methods used has ever been made which was not fully appreciated by the Survey engineers; and they have made combined effort to correct, as far as possible, each defect in the work as soon as it is discovered. The instruments and methods have been subject to continual change, and old have been supplanted by new whenever the new have been proved the better. Constructive criticism of the work is, and has always been, welcomely received and acted upon, and this opportunity is taken to appeal to engineers who are interested in the use of stream flow records for more constructive criticism and for co-operation in raising this branch of hydraulics to the high plane for which all who are engaged in it are striving.

A few points in Mr. Groat's paper may be referred to briefly:

1.—The paper calls attention to the recognized fact that the differential meter is affected by cross-currents.

2.—It is believed by the speaker that the methods used in carrying on the work from which the conclusions in the paper were drawn, and the magnitude of the results obtained, are both open to question.

3.—Unfortunately, the methods are described only in a general way, and various factors which might affect the results materially are

not adequately presented. Furthermore, sufficient data for making any exhaustive study of the results are not given. Therefore, any detailed discussion of the work is impossible. Mr. Hoyt.

4.—The large Price meter used in the work is of a type which has not been used for several years by engineers who are most familiar with stream gauging. Its use in the work of the Geological Survey was abandoned several years ago, when fifty meters of this style, all practically new, were condemned and replaced by the small Price meter, which was found to give more satisfactory results.

5.—The reader is left in great doubt and uncertainty in regard to the standards used in the comparisons. Though the Haskell meter used may not be seriously affected by cross-currents, there are no experimental data, as far as known to the speaker, which show that this meter gives results any more accurate under the adverse conditions stated than does the Price meter.

6.—The results obtained with an unrated Pitot tube do not appear to be sufficiently conclusive for any final deductions. The Pitot tube itself has had no extended use in open channels, its behavior has not been fully investigated, and, in the limited experiments which have been made with it, many questions have come up which leave an uncertainty as to its action. It has been demonstrated that its rating does not hold near the sides of a canal, and it has also been shown that it will not measure the true velocity when the opening is not in line with the current.

7.—The paper points out a well-known defect in the Price meter, but gives no remedy, and offers no substitute which can be used under a wide variety of conditions.

8.—The assumption that the differences shown by the so-called still-water ratings of the meters apply when the meters were used in the canal is questionable. The amount of difference between the Price and Haskell meters, when rated under the conditions mentioned, will depend on the amount of disturbance introduced into the rating; as the Price meter will record, not only the horizontal motion, but also the vertical motion and the cross-currents.

9.—The experience of the Geological Survey shows that accurate ratings of the current meter cannot be made from boats, owing to the errors introduced in connection with obtaining the various factors entering into the rating. Changes in the methods of rating by the United States Geological Survey, the Bureau of Standards, and the Rensselaer Polytechnic Institute have changed the results of ratings about 3 per cent.

10.—The speaker is unable to see why one familiar with current-meter work would have attempted to use the Price current meter under the conditions found at Massena. Engineers of the Geological Survey have frequently refused to co-operate in work where similar conditions existed.

Mr. Hoyt. 11.—The statistical synthesis method, as understood by the speaker, does not take into account the relative distribution of velocities and areas in various parts of the cross-section. The mean velocity in a cross-section, as defined by the Geological Survey, is obtained by dividing the discharge by the area, which will not give the same results as taking the mean of the velocities at various points in the cross-section. For example, a certain stream may have 50% of its flow in 10% of its cross-section. Furthermore, this method of computation has no apparent advantage over that ordinarily used, and has the disadvantage of being much more complicated, even if the results are of equal value.

12.—The question of the distribution of velocity in the vertical has been investigated most thoroughly by the engineers of the Geological Survey, who have found the mid-depth method to be the least desirable. The mean of the velocities at two-tenths and eight-tenths of the depth gives (without a coefficient) the velocity in the vertical, under all conditions of natural flow; and, next in value, the six-tenths depth method has been found to be the most satisfactory.

The only method yet devised for rating a current meter is by drawing it through still water. A close study indicates that no material error is introduced in the results obtained with this instrument on account of the method of rating. Large numbers of comparative measurements have been made with the Price current meter, calibrated weirs, and measuring tanks, all of which show that the meter, when properly used, will give results accurate within 1 or 2 per cent

It has also been found that measurements at different points on a stream, where the conditions of measurement are entirely different, give results which agree closely. Many measurements have been made on two tributary streams and then on the main stream below, and the results check within the limits of accuracy necessary in work of this type.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

THOMAS CHALKLEY JAMES BAILY, JR., M. Am. Soc. C. E.*

DIED DECEMBER 6TH, 1912.

Thomas Chalkley James Baily, Jr., was born in Boston, Mass., on December 20th, 1867. His father, Major Thomas Chalkley James Baily, U. S. A., served throughout the Civil War, being commissioned First Lieutenant on May 14th, 1861, and retiring with the rank of Major, on August 18th, 1865. His mother, Carrie E. Todd, a native of Augusta, Me., was of Puritan ancestry, and descended from the earliest New England settlers.

After attending the local schools, Mr. Baily prepared for college at the Newark School, Newark, Del. He afterward entered Lehigh University, and was graduated in the Class of 1890, with high honors, and with the degree of Civil Engineer.

For two years after his graduation, he was a member of the Engineer Corps of the Erie Railroad, stationed at New Castle, Pa., resigning to accept a position on the Mississippi River improvement, where he was engaged in examinations and surveys of the mouth of the Mississippi. In 1893, he entered the Engineer Department of the District of Columbia, first as Draftsman and then as Assistant Engineer in the Highway Division. In 1908, he was appointed Principal Assistant Engineer in the Sewer Department of the District of Columbia; two years later he was made Assistant Engineer in charge of Street Extensions; and, in 1910, he was appointed Engineer of Bridges for the District of Columbia.

Mr. Baily was in charge of the construction of the Anacostia River Bridge, and, as Engineer of Bridges, he designed and built a number of important structures, among them the Cedar Street Subway at Takoma Park, D. C., and the Belmont Street Improvement. At the time of his death, he had just completed the plans and specifications for the Q Street Viaduct, crossing Rock Creek Valley at Washington.

He assisted in the design and construction of the Edmonton Avenue Bridge at Baltimore, Md., and collaborated with W. J. Douglas, M. Am. Soc. C. E., in writing the chapters on Masonry and Timber

* Memoir prepared by Asa E. Phillips, M. Am. Soc. C. E.

Structures for the "American Civil Engineers' Pocket Book," edited by Mansfield Merriman, M. Am. Soc. C. E.

At the time of his death, Mr. Baily was making investigations of the Harbor of Progresso, Yucatan, for Barclay Parsons and Klapp, Consulting Engineers. He died suddenly in the City of Mexico, on December 6th, 1912.

In 1894, he was married to Miss Mary Rodgers, of Orange, N. J., who, with one daughter and one son, survives him.

Mr. Baily was a thorough and painstaking engineer, and an exceptionally devoted student of his Profession. He had a kind and genial nature, and was esteemed and admired by all who knew him. He was a devoted father, a faithful friend, and a good citizen.

Mr. Baily was elected a Member of the American Society of Civil Engineers on October 4th, 1905.

CHARLES LEWIS HARRISON, M. Am. Soc. C. E.*

Died September 14th, 1912.

Charles Lewis Harrison was born on his father's farm at Auxvasse, in Callaway County, Missouri, on March 5th, 1857, and was one of a family of twelve, having seven sisters and four brothers. He was descended from the families of Harrison and Crockett, of Virginia, his grandfather, Major John Harrison, emigrating to Missouri in 1817, and being one of the earliest American settlers in Callaway County. His parents were Thomas Harrison and Catherine (Maddox) Harrison. Thomas Harrison was born in Virginia, and moved with the family to Missouri when he was less than a year old; he served as a volunteer during the war with Mexico; tried mining in Michigan for several years in the Thirties; went to California in 1849, but returned to Auxvasse in a few years and passed the remainder of his life in Callaway County, where he became a prosperous farmer and a man of wide influence in the community, exhibiting those qualities of helpfulness and thought of others so strongly marked in his son Charles.

Mr. Harrison's early education was acquired in the public schools, from which, in 1876, he entered the University of Missouri, receiving the degree of T. E. in 1879 and the degree of C. E. in 1880. During the first two years of his course he was a working student, his father refusing to help him until, as he expressed it, he found out "what the boy was made of" and what he wanted to do. In college he exhibited those qualities for thoroughness, ready debate, and argument, which were so marked throughout his life.

* Memoir prepared by Alfred Noble, Past-President, Am. Soc. C. E., and J. Waldo Smith, M. Am. Soc. C. E.

After being graduated from the University, he entered the service of the Mississippi River Commission as a Rodman, and assisted in measuring the discharge of the Mississippi at Fulton, Tenn., until August, 1880, when he was transferred to Grafton, Ill., with the grade of Recorder, continuing in the work of discharge measurements until October, 1881. He was then transferred to Lake Providence, La., as Assistant on hydrographical and topographical surveys. He was obliged to give up this work in June, 1882, on account of malaria, but was offered employment under the Corps of Engineers of the Army, and from June to December, 1882, was Transitman on a topographical survey of the Missouri River from Fort Pierre to Fort Randall, Dak.; from December, 1882, to September, 1883, he was Instrumentman on a topographical survey of the Red River of the North.

The construction of reservoirs at the head-waters of the Mississippi was then in progress, under the direction of the Army Engineers, and Mr. Harrison was engaged as Assistant in the construction of a dam across that river at Pokegama Falls, Minn., from September, 1883, to July, 1884, and as Engineer in charge of the completion of a dam at the outlet of Lake Winnebigoshish, Minn., during the remainder of the year. During the five years ensuing he was with an uncle in Chicago, in the live stock commission business.

From July, 1890, to May, 1891, Mr. Harrison took part as Assistant Engineer in hydrographical surveys and investigations for the Sanitary District of Chicago; he then went to Northern Michigan as Engineer in charge of the improvement of Carp River and of the construction of a sewerage system for the City of Ishpeming, returning in February, 1892, to the Sanitary District to take charge of surveys for the location of the Chicago Drainage Canal, and from August, 1892, to August, 1897, was Division Engineer in charge of construction of Sections 5 to 15, embracing excavation of various kinds on a large scale and much concrete and structural work, including, with other important features, the regulating works at Lockport, Ill. While with the Sanitary District, Mr. Harrison served under Lyman E. Cooley and Isham Randolph, Members, Am. Soc. C. E., successively Chief Engineers. This was the first opportunity afforded to show to the Engineering Profession his ability as an engineer and an executive, and his work gave him a high standing.

In the summer of 1897, the United States Board of Engineers on Deep Waterways was organized to make surveys and estimates for a deep waterway from the Great Lakes to tide-water, and Mr. Harrison was the first engineer engaged. As Division Engineer he had charge of two sections, one being the Niagara Ship Canal route, from Lake Erie to Lake Ontario, the other extending from Troy to Lake Champlain. On the completion of this work at the end of 1900, he went to Panama for the Isthmian Canal Commission, where he made sur-

veys to verify the work of the engineers of the French Company and also made borings at the site of the Bohio Dam and at other points. From June, 1900, to May, 1902, he was Chief Engineer for the Denver Union Water Company, and designed and built the Lake Cheesman Dam, notable as sustaining a greater head of water than any other structure then existing, and without any leakage.

At the end of May, 1902, Mr. Harrison accepted the position of Principal Assistant Engineer of the East River Division of the New York Terminal of the Pennsylvania Railroad. The division included four tunnels under the East River, built in compressed air, this class of work continuing nearly three years. In April, 1908, he suffered an injury of the heart from too long a stay under pressure, from which he never recovered. The work was then so nearly completed that if he could have endured for another day, he would not have incurred any further risk. He attempted to resume professional work in April, 1909, as Deputy Chief Engineer for the Board of Water Supply of the City of New York, but after a year's trial was compelled to give it up, and from that time limited himself to consultations. He passed the winter of 1910-11 in Southern California; in the winter of 1911-12 he was in Havana and was greatly interested in the raising of the battleship *Maine*, then in progress. In March, 1912, he visited the Panama Canal.

In the latter part of August, 1912, he contracted a slight cold which did not take a serious form until two or three days before his death. He was buried in the family plot in the churchyard of the Presbyterian Church at Auxvasse.

Mr. Harrison's engineering experience embraced a wide range of successful work. To every new problem he gave the most thorough study, from the first step to the completion, and his judgment thus founded and trained was seldom or never erroneous. During his many years of participation in important work, he trained and developed a large number of young engineers from whom he required the same thoughtfulness and thoroughness, and nothing was more certain to incur his criticism than lack of care or misdirected energy, but it was given so kindly that no bitterness followed, and he possessed the friendship and loyalty of his assistants to a remarkable degree.

His work at three important industrial centers—Chicago, Denver, and New York—had made him well known throughout the United States as a capable engineer, an executive of rare ability, and as a man of the highest integrity. That his further career would have been most distinguished, if his health had permitted, is evidenced by the large number of engagements he was obliged to decline.

He loved the truth in all things, and hated any form of deceit or misrepresentation; he was generous to a fault, but for the hypocrite or professionally impecunious he had no consideration. His patience and

perseverance, as well as his thoughtfulness for, and kindliness to, those associated with him, was particularly marked, and he was always a most loyal friend.

His recreation and hobby was a most unusual one, in that he was the only engineer, or one among a few, who owned a racing stable. His stable, although small, was chosen with the same good judgment and attention to detail shown in his engineering work, so that his colors were often in the lead on the principal tracks of the country and his percentage of winners was greater than in the stables of the wealthy owners.

Mr. Harrison was elected a Member of the American Society of Civil Engineers on March 2d, 1898, and was a Director from 1908 to 1910. He was a frequent contributor to the discussions on papers before the Society; in 1905, the Thomas Fitch Rowland Prize was awarded to him and S. H. Woodard, M. Am. Soc. C. E., as joint authors, for their paper entitled "Lake Cheesman Dam and Reservoir."*

He was also a member of the Western Society of Engineers, and of the University and Engineers' Clubs of New York City.

JOHN HAWKESWORTH, Assoc. M. Am. Soc. C. E.†

Died December 10th, 1912.

John Hawkesworth, the son of James A. and Ella Zebley Hawkesworth, was born in New York City, on May 18th, 1883. He prepared for college at the Cutler School, and, in 1900, entered Columbia University, from which he was graduated in 1904, with the degree of C. E.

In May, 1904, Mr. Hawkesworth entered the employ of Raymond F. Almirall, M. Am. Soc. C. E., as Assistant Engineer, and was engaged on the design and superintendence of construction of various reinforced concrete buildings and structures in and around New York City. From 1906 to 1908, he served as Business Manager for Mr. Almirall and had general supervision of the work of the office.

Mr. Hawkesworth was one of the first engineers to engage in structural tests of reinforced concrete, and the results of his experiments were published in 1904 in "Cements, Mortars, and Concretes," by Myron S. Falk, M. Am. Soc. C. E. Other articles on this subject were written by Mr. Hawkesworth and published in *Engineering News* in 1905. He was also the author of a "Graphical Handbook for Reinforced Concrete Design," which was issued in 1906, and of various

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 89.

† Memoir prepared by Milton L. Cornell, Esq.

other technical books and papers, one of the latter being entitled "Precarious Expedients in Engineering Practice," presented before this Society on February 16th, 1910.*

From 1908 to 1912, Mr. Hawkesworth spent part of his time in travel abroad and part in consulting engineering practice in this country. He was also engaged in investigating corporations desirous of increasing their capital. Shortly before his death he organized The Carleton Company, Engineers and Contractors, of New York City, of which he was President and also a Director.

Mr. Hawkesworth was married, on November 25th, 1911, to Miss Marion Everett Barling who survives him.

He was a member of the Lotus Club, the Columbia University Club, the Westchester Country Club, the Municipal Art Society, and of Psi Upsilon.

Mr. Hawkesworth was elected a Junior of the American Society of Civil Engineers on September 6th, 1904, and an Associate Member on November 4th, 1908.

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* *Transactions*, Am. Soc. C. E., Vol. LXVII, p. 32.

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JONATHAN P. SNOW	EMIL GERBER	LEONARD METCALF
ROBERT RIDGWAY	WILLIAM CAIN	HENRY R. LEONARD
LEONARD W. RUNDLETT	E. C. LEWIS	EDWARD H. CONNOR
WILLIAM H. COURTENAY	W. A CATTELL	SAMUEL H. HEDGES

Assistant Secretary, T. J. McMINN

Standing Committees

(The President of the Society is *ex-officio* Member of all Committees)

On Finance:	*On Publications:*	*On Library:*
LINCOLN BUSH	JAMES H. EDWARDS	J. WALDO SMITH
GEORGE C. CLARKE	ROBERT RIDGWAY	CHARLES D. MARX
HENRY W. HODGE	CHARLES S. CHURCHILL	T. KENNARD THOMSON
LEONARD METCALF	WILLIAM CAIN	E. C. LEWIS
EMIL GERBER	JONATHAN P. SNOW	CHAS. WARREN HUNT

Special Committees

On Concrete and Reinforced Concrete: Joseph R. Worcester, J. E. Greiner, W. K. Hatt, Olaf Hoff, Richard L. Humphrey, Robert W. Lesley, Emil Swensson, A. N. Talbot.

On Engineering Education: Desmond FitzGerald, Onward Bates, D. W. Mead.

On Steel Columns and Struts: Austin L. Bowman, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

On Bituminous Materials for Road Construction: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

On Valuation of Public Utilities: Frederic P. Stearns, H. M. Byllesby, Thomas H. Johnson, Leonard Metcalf, Alfred Noble, William G. Raymond, Jonathan P. Snow.

To Investigate Conditions of Employment of, and Compensation of, Civil Engineers: Alfred Noble, S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, John A. Bensel.

To Codify Present Practice on the Bearing Value of Soils for Foundations, etc.: Robert A. Cummings, Edward C. Shankland, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner, Frank M. Kerr.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

House of the Society—220 West Fifty-seventh Street, New York.

Telephone Number....................5913 Columbus.

Cable Address...................."Ceas, New York."

Vol. XXXIX. APRIL, 1913. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS
OF THE SOCIETY

March 19th, 1913.—The meeting was called to order at 8.40 P. M.; Director T. Kennard Thomson in the chair; Charles Warren Hunt, Secretary; and present, also, 130 members and 19 guests.

A paper by E. J. Schneider, M. Am. Soc. C. E., entitled "Construction Problems, Dumbarton Bridge, Central California Railway," was presented by the Secretary, and illustrated with lantern slides.

The Secretary also read a communication on the subject from L. J. Le Conte, M. Am. Soc. C. E., and the paper was discussed orally by Messrs. James B. French, J. A. Knighton, James Owen, O. E. Hovey, L. D. Rights, R. D. Coombs, and T. Kennard Thomson.

The Secretary announced the following deaths:

Philip Henry Coombs, elected Member, March 7th, 1906; died March 6th, 1913.

Charles Spearman, elected Associate Member, October 5th, 1909; died August, 1912.

Adjourned.

April 2d, 1913.—The meeting was called to order at 8.35 P. M.; President George F. Swain in the chair; Chas. Warren Hunt, Secretary; and present, also, 140 members and 12 guests.

The minutes of the meetings of February 19th and March 5th, 1913, were approved as printed in *Proceedings* for March, 1913.

A paper by Lewis J. Johnson, M. Am. Soc. C. E., and John R. Nichols, Jun. Am. Soc. C. E., entitled "Shearing Strength of Construction Joints in Stems of Reinforced Concrete T-Beams, as Shown by Tests," was presented by Mr. Nichols.

The Secretary read a communication on the subject from L. J. Mensch, M. Am. Soc. C. E., and the paper was discussed orally by Messrs. J. P. Snow, David Gutman. E. E. Seelye, A. B. Heiser, H. G. Raff, G. C. Doyen, and T. H. Wiggin.

A paper by Joshua Fielden Ramsbotham, Assoc. M. Am. Soc. C. E., entitled "Fremantle Graving Dock: Steel Dam Construction for North Wall," was presented by title. The Secretary read a communication on the subject from A. R. Archer, Assoc. M. Am. Soc. C. E.

The Secretary announced the election of the following candidates on April 2d, 1913:

As Members

Alfred Victor Bouillon, Seattle, Wash.
Allan Theodore Dusenbury, New Orleans, La.
Joseph Wilton Ellms, Cincinnati, Ohio
Joseph Dean Evans, Montreal, Que., Canada
Alexander Gray, Ottawa, Ont., Canada
Herbrand Harvey, St. Johnsville, N. Y.
Henri Herbert Henderson, Stockton, Cal.
George B Herington, La Fayette, La.
Alfred Elmer Hess, Williamsport, Pa.
Owen Meriwether Jones, Sault Ste. Marie, Mich.
George Bertram de Betham Kershaw, West Wickham, England
Felix John Kersting, Leavenworth, Kans.
Stuart Kelsey Knox, East Orange, N. J.
Victor Hugo Kriegshaber, Atlanta, Ga.
Arthur Barrett Miller, New York City
John Portman Paget, Guayaquil, Ecuador
Lewis Frederick Patstone, Ada, Ohio

As Associate Members

Frank Hicks Adams, Wiggins, Colo.
Juan Batiste Hipolyte Bardury, Guayama, Porto Rico
Dudley Seymour Bright, Pittsburgh, Pa.
Arthur Stacey Buss, High Falls,. N. Y.
James Retzer Comly, San Diego, Cal.
Robert Emmet Cullen, Peak, S. C.

Robert Curtis Cutting, Hogsett, W. Va.
Arthur Albert Davis, Bethlehem, Pa.
Ernest Buel Day, New York City
Charles Thomas Delamere, Port Arthur, Ont., Canada
Harry Jocelyn Dignum, Preston, Oriente, Cuba
William Frederick Farley, Montreal, Que., Canada
Gustaf Adolf Flink, Harrisburg, Pa.
Norman Paul Gerhard, Scarsdale, N. Y.
Henry McCormick Gross, Harrisburg, Pa.
Frederic Hamilton Hill, Wilmington, Del.
James Carey Jordan, Pittsburgh, Pa.
Henry Larsen, Portland, Ore.
Ralph Jordan Lawrence, Philadelphia, Pa.
James Bennett Lowell, Worcester, Mass.
Joseph Davis Metcalfe, Caldwell, Tex.
James Blaine Miller, Washington, D. C.
Edgar Henry Mix, Venice, Cal.
James Alexander Moffat, Spuzzum, B. C., Canada
Sherman Moore, Detroit, Mich.
Everett Bodman Murray, Kansas City, Mo.
William Hogarth Robertson Nimmo, Vancouver, B. C., Canada
George Bushnell Palmer, New York City
John Louis Pickles, Duluth, Minn.
Raymond Edgar Reynolds, Buffalo, N. Y.
Frederick Charles Scobey, Washington, D. C.
Edward Lewis Shepard, East Lansing, Mich.
Harry Evans Sovereign, Grand Valley, Colo.
Harry Stock, Chicago, Ill.
Frank Thorn Townsend, Buffalo, N. Y.
Haswell Roger Williams, Baltimore, Md.
Harry Dugan Williar, Jr., Baltimore, Md.
Frederick Carl Youngblutt, Savage, Mont.

As Associate

Herbert Chase Titus, Albany, N. Y.

As Juniors

Robert Crewdson Benson, Tatura, Victoria, Australia
Harold Edwin Curtis, St. Albert, Alta., Canada
Rex Edward Edgecomb, Corvallis, Ore.
Harold Bernard Hammill, Berkeley, Cal.
Adolf Hinrichs, Brooklyn, N. Y.
Carl Wayne Mengel, Norfolk, Va.
Robert Ansley Monroe, Berkeley, Cal.
Alec Alfred Plummer, Vancouver, B. C., Canada
Horatio Seymour, Jr., Storm King, N. Y.

The Secretary announced the transfer of the following candidates on April 2d, 1913:

FROM ASSOCIATE MEMBER TO MEMBER

WALTER CHEW BARTON, New Orleans, La.
LUTHER HAROLD BURT, Hartford, Conn.
JOHN SOULE BUTLER, Nashville, Tenn.
GEORGE ALBERT HAYNES, Waukesha, Wis.
CHARLES WILSON KILLAM, Cambridge, Mass.
JOHN MARTIN, New York City
OSCAR CHARLES MERRILL, San Francisco, Cal.
JAMES BOORMAN STRONG, Hillburn, N. Y.

FROM ASSOCIATE TO MEMBER

EDWARD MICHAEL GRAVES, Cleveland, Ohio

FROM JUNIOR TO ASSOCIATE MEMBER

JAMES NORTH EDY, Billings, Mont.
WALLACE HAYNES HALSEY, Bridgehampton, N. Y.
JOHN GIBSON HENDRIE, Brighton, Trinidad
GRANVILLE JOHNSON, Jamaica Plain, Mass.
GEORGES PIERRE FERDINAND JOUINE, Vicksburg, Miss.
GEORGE GLENN MCDANIEL, San Francisco, Cal.
FRANK JOHNSON TRELEASE, Buffalo, N. Y.

The Secretary announced the following deaths:

PETER SUTHER ARCHIBALD, elected Member, January 7th, 1885; died March 16th, 1913.

CHARLES KELLOGG, elected Member, June 2d, 1880; died March 23d, 1913.

EMIL EDWARD KUERSTEINER, elected Member, December 1st, 1897; died March 10th, 1913.

JOHN STUART ELLIOTT, elected Associate Member, April 6th, 1892; died March 25th, 1913.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

March 4th, 1913.—President Swain in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bates, Bush, Clarke, Gerber, Hodge, Leonard, Metcalf, Ridgway, Smith, Snow, Thomson, and Wallace.

The proposed re-districting of the Society for the purpose of the Nominating Committee was considered in a report received from a sub-committee appointed by the Board for the purpose, and the proposed amendment, as prepared by that Committee, was approved, and the Committee instructed to secure signatures enough to bring the matter legally before the Society.*

The Secretary reported that Messrs. Chas. D. Marx, W. A. Cattell, Arthur L. Adams, and Chas. Derleth, Jr., have accepted their appointment by the Board, to represent this Society on the General Committee in charge of The International Engineering Congress, 1915.

The application of a woman for admission to the Society was declined, for the reason that the Board is of the opinion that the Constitution of the Society does not contemplate the admission of women.

The resignations of 1 Member and 3 Associate Members were accepted, and the resignation of 1 Associate was accepted as taking effect December 31st, 1913.

Ballots for membership were canvassed resulting in the election of 16 Members, 23 Associate Members, and 11 Juniors, and the transfer of 10 Juniors to the grade of Associate Member.

Nine Associate Members were transferred to the grade of Member.

Applications were considered, and other routine business transacted.

Adjourned.

April 2d, 1913.—President Swain in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Edwards, Endicott, Gerber, Leonard, Macdonald, Metcalf, Ridgway, Smith, Snow, and Thomson.

The Finance Committee presented a report covering the work of the employees of the Society and their salaries, which was received and ordered published in *Proceedings* for the information of the membership.†

A letter-ballot of the whole Board (ordered February 4th, 1913) on the question as to whether the salary of the Secretary shall be increased from $10 000 to $12 000 per annum was canvassed. Every member of the Board, except the Secretary, cast a ballot, with the result that the question was decided in the affirmative.

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* This amendment has been mailed to all members.

† This report will be found on page 285.

A telegram addressed to the Chairman of the Assembly Committee on Public Education at Albany, N. Y., protesting against the passage of Assembly Bill No. 1126 for the Licensing of Civil Engineers was adopted and forwarded.

The President was requested to write to the Governor of New York, calling attention to the attitude of this Society on the proposed Bill, and on the general subject of the Licensing of Engineers.

Ballots for membership were canvassed, resulting in the election of 17 Members, 38 Associate Members, 1 Associate, and 9 Juniors, and the transfer of 7 Juniors to the grade of Associate Member.

Eight Associate Members and 1 Associate were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

REPORT OF THE FINANCE COMMITTEE TO THE BOARD OF DIRECTION OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS*

Received April 2d, 1913.

The Finance Committee reports that it has conferred with the Secretary regarding the number of employees of the Society, the amount and character of the work performed by each, and the salaries paid to them; and has visited the various departments during working hours, and inspected the work being done.

The Committee was impressed with the large amount of detailed work required to successfully conduct the work of the Society, maintain its library and records, and render prompt service to its members. It is of the opinion that the work is being efficiently and economically done, that the number of employees now upon the payroll is necessary to properly handle the work, and that the salaries paid are consistent with the character and amount of work accomplished.

The following is a list of the present employees, with their salaries:

	Per Annum.
Office Assistant	$2 800
Ass't. Editor	1 080
Bookkeeper	1 380
Ass't. Bookkeeper	660
Membership Clerk	1 080
Stenographer	900
"	900
"	780
"	780
Address Clerk	780
" "	780
Clerk	720
Typist	480
Mailing Clerk	720
Junior Clerk	408
" "	336
" "	252
" "	252
" "	252
Hall Boy	252
" "	156
Librarian	2 100
Ass't. Librarian	1 080
" "	900
" "	900
" "	660
Janitor	720
Ass't. Janitor	468
	$22 576

* Published for the information of members, by order of the Board.

The Society also has the following salaried officers:

	Per Annum.	
Secretary	$10 000	
Ass't. Secretary	3 000	
Treasurer	100	$13 100

These salaries are divided by departments as follows:

GENERAL:		
Secretary	$10 000	
Treasurer	100	$10 100
EDITORIAL:		
Ass't. Secretary	$3 000	
Ass't. Editor	1 080	4 080
LIBRARY:		
Librarian	$2 100	
4 Ass't. Librarians	3 540	5 640
CARETAKING:		
Janitor and Ass't	$1 188	
2 Hall Boys	408	1 596
GENERAL OFFICE:		
Chief Office Assistant		2 800
Membership:		
2 Clerks		1 800
Bookkeeping:		
2 Bookkeepers		2 040
Office Work:		
4 Stenographers	$3 360	
2 Address Clerks	1 560	
1 Typist	480	
1 Mailing Clerk	720	
5 Junior Clerks	1 500	7 620
		$35 676

LINCOLN BUSH,
GEORGE C. CLARKE,
HENRY W. HODGE,
LEONARD METCALF,
EMIL GERBER,
GEORGE F. SWAIN, *Ex-officio,*
Finance Committee.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

May 7th, 1913.—8.30 P. M.—A regular business meeting will be held, and two papers will be presented for discussion, as follows: "Colorado River Siphon," by George Schobinger, Jun. Am. Soc. C. E.; and "Tidal Phenomena in the Harbor of New York," by H. de B. Parsons, M. Am. Soc. C. E.

Mr. Schobinger's paper was printed in *Proceedings* for March, 1913, and Mr. Parsons' paper is printed in this number of *Proceedings*.

May 21st, 1913.—8.30 P. M.—Two papers will be presented for discussion at this meeting, as follows: "Recent Improvements in Leveling Instruments," by Dunbar D. Scott, M. Am. Soc. C. E.; and "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors," by John R. Nichols, Jun. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

June 4th, 1913.—8.30 P. M.—This will be a regular business meeting. A paper by Maurice G. Parsons, Jun. Am. Soc. C. E., entitled "The Philosophy of Engineering," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-fifth Annual Convention of the Society will be held at Ottawa, Ont., Canada, from June 17th to 20th, 1913, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION

Charles H. Rust, *Chairman,*

Henry W. Hodge, Chas. Warren Hunt.

LOCAL COMMITTEE

Chas. H. Keefer, *Chairman,*

W. H. Breithaupt,	H. Holgate,	John Kennedy,
S. J. Chapleau,	J. A. Jamieson,	William McNab,
C. R. F. Coutlee,	Phelps Johnson,	C. H. Mitchell,
A. R. Dufresne,	T. C. Keefer,	H. R. Safford,
G. H. Duggan,	H. G. Kelley,	W. F. Tye,
Sir Sandford Fleming,		G. W. Volckman,

A preliminary circular has been sent to all members, and, as soon as arrangements have been completed, an additional circular will be issued.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and, on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

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* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Gavin N. Houston, M. Am. Soc. C. E., 409 Equitable Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, and, until further notice, will take place at the Colorado Traffic Club.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association

On March 14th, 1912, the Atlanta Association of Members of the American Society of Civil Engineers was organized, with the following officers: Arthur Pew, President; William A. Hansell, Jr., Secretary; and Messrs. James N. Hazlehurst and Alexander Bonnyman, Members of the Executive Committee. The Association will hold its meetings in the house of the University Club.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 302 Board of Trade Building, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From March 5th to April 1st, 1913)

DONATIONS*

RIVER DISCHARGE.

Prepared for the Use of Engineers and Students. By John Clayton Hoyt, M. Am. Soc. C. E., and Nathan Clifford Grover, Assoc. M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9¼ x 6 in., illus., 12 + 173 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1912. $2.00.

In this book, the authors, it is stated, have considered the methods of measuring and computing stream flow, the laws which govern such measurements, the phenomena which affect the flow of streams, and the uses to which the data can be applied. The first edition was published in 1907 since which time material advance, it is said, has been made in the development of methods and instruments for stream gauging due to the work of the engineers of the Water Resources Branch of the United States Geological Survey. The Survey's methods have been accepted as standards in this country, it is stated, and have been adopted and used in hydraulic developments by engineers in all parts of the world. In this, the second edition, the authors state that they have attempted to incorporate the latest practice in the subject, the chapter on instruments and equipment having been wholly rewritten and that on the compilation and use of data having been revised and enlarged. Other minor changes and additions have been made, the latter including maps showing the mean annual precipitation and run-off in the United States. The Contents are: Introduction; Instruments and Equipment; Velocity-Area Stations; Weir Stations; Discussion and Use of Data; Conditions Affecting Stream Flow; Tables; Index.

EARTHWORK HAUL AND OVERHAUL.

Including Economic Distribution. By J. C. L. Fish, M. Am. Soc. C. E. Cloth, 9¼ x 6 in., illus., 14 + 165 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1913. $1.50.

All questions relating to the computation of overhaul and the use of the mass curve in planning distribution, are, the preface states, answered in this book. The subject-matter is divided into two parts. Part I, Haul and Overhaul, is planned for the use of students and teachers, railroad contractors, computers, and railroad engineers. In this part, Chapters I to V are recommended to students of overhaul, in school or out, as they contain, it is said, a full presentation of each of the elements of overhaul computation. Descriptions of bases and methods of overhaul computation are given in Chapters V, VI, and VII, which, it is stated, railroad contractors will find useful in coming to a definite understanding with railroad engineers as to the way in which overhaul shall be computed. Chapters VI and VII may also be of use to railroad engineers and computers in determining a method of computation and a complete plan of procedure under such method. Part II, Economic Distribution of Material Along the Profile, is devoted, the preface states, to the elements of the problem of economic distribution and presents a thorough treatment of the solution of this problem by the use of the mass curve. The Chapter headings are: Part I, Haul and Overhaul: Considerations Preliminary to the Computation of Haul; The Mass Curve; Limits and Center of Mass of a Body of Material; Center of Gravity; Overhaul, Free Haul, and Cross Haul; Overhaul Computed for the Simple Case of Fig. 19; Overhaul Computed for the Complex Case of Fig. 38. Part II, Economic Distribution of Material Along the Profile: Preliminary Considerations; Economic Balancing Line for Mass Curve Plotted from Cut-Volumes and Equated Fill-Volumes, etc.; Economic Balancing Line for Mass Curve Plotted from Fill-Volumes and Equated Cut-Volumes; Index.

RATIONAL *AND* APPLIED MECHANICS.

By Calvin Milton Woodward. Cloth, 9½ x 6¼ in., illus., 8 + 517 pp. St. Louis, Mo., Nixon-Jones Printing Company, 1912. $4.00. (Donated by the Author.)

It is stated in the preface that this book was written primarily for students entering upon their second collegiate year. Their knowledge of mechanics being limited to what they have gained during their study of physics and from practice in the laboratory and shop, the author aims to take them by logical steps from

* Unless otherwise specified, books in this list have been donated by the publishers.

the purely elementary to the higher mathematical treatment of the subject and to make every step reasonable and every demonstration intelligible, in order that they may learn to follow statements made in the language of mathematics and mechanics. The subject-matter, it is stated, has not been confined to any one branch of Engineering or Architecture, the object being to make it possible for the student in any technical branch to read intelligently and use carefully prepared papers and manuals and to solve new problems as they arise. No attempt has been made to multiply problems, clear illustrations of general principles and useful methods of analysis only being included. The author states that he has used abstract problems freely with those which approximate more or less closely real ones. With the abstract problems he has aimed to illustrate general methods, establish general laws, and derive general formulas and then apply them to the solutions of problems derived from all sorts of sources and conditions. Matter relating to commercial and cost data has been omitted, and the author has adopted that order for his subjects, it is stated, which will meet the needs of the engineering student who early in his professional study takes up frames and structures. The Contents are: Introduction, Part I, Statics: Co-Linear Forces; Parallel Forces Which Balance; Converging Forces in a Plane; Non-Converging Forces in a Plane; Forces in Space; Centroids of Surfaces; Centres of Gravity; Moments of Inertia of Surfaces; Elementary Graphical Statics; Internal Stress. Part II, Kinetics: Translation Under Constant Forces; Translation Under Varying Forces; Moments of Inertia of Solids; Deviating Forces; Kinematics; Work and Energy; Elasticity; Graphical Representations of Shear, etc.; Shearing Stress; Beams of Uniform Strength; Work in a Bent Beam, etc.; The Stability of Foundations and Retaining Walls; Eccentric Loading of Short Columns; The Energy of Streams of Water and Air; The Efficiency of Compressed Air; Appendix: Four-Place Logarithms; Index.

PSYCHOLOGY AND INDUSTRIAL EFFICIENCY.

By Hugo Münsterberg. Cloth, $8\frac{1}{4}$ x $5\frac{1}{4}$ in., 8 + 321 pp. Boston and New York, Houghton Mifflin Company, 1913. $1.50.

The author's aim in this book, it is stated, has been to sketch the outlines of a new science which is intermediate between the modern laboratory psychology and the problems of economics, and to demonstrate the principles and methods of such experimental economic psychology by characteristic illustrations. To that end he has selected for discussion herein three central purposes which occur in every department of business life, namely (1) how to find the man whose mental qualities make him best fitted for the work; (2) under what psychological conditions can the greatest and most satisfactory output of work be secured from every man; and (3), how completely can the influences on human minds which are desired in the interest of business, be produced. The Contents are: Introduction; Applied Psychology; The Demands of Practical Life; Means and Ends. I, The Best Possible Man: Vocation and Fitness; Scientific Vocational Guidance; Scientific Management; The Methods of Experimental Psychology; Experiments in the Interest of Electric Railway Service; Experiments in the Interest of Ship Service; Experiments in the Interest of Telephone Service; Contributions from Men of Affairs; Individuals and Groups. II, The Best Possible Work: Learning and Training; The Adjustment of Technical to Psychical Conditions; The Economy of Movement; Experiments on the Problem of Monotony; Attention and Fatigue; Physical and Social Influences on the Working Power. III, The Best Possible Effect: The Satisfaction of Economic Demands; Experiments on the Effects of Advertisements; The Effect of Display; Experiments with Reference to Illegal Imitation; Buying and Selling; The Future Development of Economic Psychology; Notes; Index.

MINING ENGINEERS' EXAMINATION AND REPORT BOOK.

By Charles Janin. Part I, Leather, $7\frac{3}{4}$ x $5\frac{1}{4}$ in., illus., 94 pp.; Part II, Boards, 5 x $7\frac{1}{4}$ in., 57 pp. San Francisco, The Mining and Scientific Press; London, The Mining Magazine, 1913. $2.50.

This work which, it is stated, has been arranged to assist prospectors, owners, promoters, etc., in presenting a concise and comprehensive report on mines and mining properties, is divided into two parts. Part I is a handbook and contains information and tables leading up to an enumeration of the points which a complete report should cover. The examination of placers is discussed, and an outline of a placer report is given as well as methods and costs of sampling with Keystone and Empire hand drills. Detailed instructions for filling out the blanks given in Part II are also included. Part II is a skeleton report and may be used as a field note-book, from which, it is stated, the final report may be readily dictated. The Contents are: Examination of Mines; Examination of Placers; Examination of Title; Blank Forms; Miscellaneous Suggestions and Information: Classification of Igneous Rocks; Study of Ore Deposits; Representative Costs; Miscellaneous Tables and Data; Index.

STABILITY IN AVIATION

An Introduction to Dynamical Stability as Applied to the Motions of Aeroplanes. By G. H. Bryan. (Macmillan Science Monographs.) Cloth, 9 x 6 in., illus., 10 + 192 pp. London, Macmillan and Co., Limited, 1911. $2.00.

The problem of stability in connection with aviation has not received, it is stated, that attention which it deserves by mathematicians, and it is hoped that, as much of the loss of life and damage to property could be avoided by a systematic study of aeroplane stability and of certain other problems regarding the motion of aeroplanes discussed in this book, its publication may lead to further study and investigation of the subject. As discussed by the author, attention is said to have been concentrated on the mathematical aspect of the problem for several reasons: (1) because the development of the mathematical theory must be done thoroughly or not at all; (2) the formulas obtained could only have been established by mathematical theory; (3) as there has seemed to be no lack of competent workers interested in the practical and experimental side of aviation, as much weight as possible should be thrown on the mathematical side of the question in order to improve the balance between theory and practice; and (4) it is hoped to advocate the claims of aeroplane equilibrium and stability as studies in colleges and universities with such branches as applied mathematics, etc. The motions or changes of motion set up in the flying machine itself by air pressures and by the other forces acting on it when equilibrium in steady motion is disturbed is, it is stated, the problem discussed and the author shows that there should be no difficulty in securing inherent stability, both longitudinal and lateral, in an aeroplane, by means of suitably placed auxiliary surfaces rigidly attached to the machine. The Chapter headings are: Introduction and Summary; Fundamental Principles; General Considerations Regarding Symmetrical Derivatives; Graphic Statics of Longitudinal Equilibrium; Longitudinal Stability of Single-Lifting Surfaces; Longitudinal Stability of Double-Lifting Systems; Asymmetric or "Lateral" Stability-Straight Planes and Vertical Fins; Lateral Stability Bent-Up Planes; General Conclusions; Comparison with Other Theories; Problems, Notes, Nomenclature, Notation; Index.

ENGINEERS' HANDBOOK ON PATENTS.

By William Macomber. Leather, 7 x 4½ in., illus., 15 + 288 pp. Boston, Little, Brown and Company, 1913. $2.50.

The engineer, the author states, is coming to be the Yankee inventor raised to the *n*th power, and as such should be familiar with the patent law. This book, it is said, is a handbook, not a textbook, on that subject, in which the author states in a practical manner, omitting legal phraseology and terminology, those things which the inventor, the industrial leader, and especially the engineer should know concerning Patent Office procedure. In it he discusses the various problems relating to patents, that is, how to obtain one, what one covers, what it protects, what must be avoided, the general course and trend of patent litigation, the problem of property rights in patents, etc. The Contents are: Introductory; What is a Patent? The Nature of Invention; What is Patentable; Patentable Novelty; The Obtaining of Patents; Claim Construction; Infringement; Patent Litigation; Property Rights; Index.

MODERN PARK CEMETERIES.

By Howard Evarts Weed. Cloth, 7¾ x 5 in., illus., 145 pp. Chicago, R. J. Haight, 1912. $1.60.

The author's aim in this book, it is stated, is to furnish information relating to the construction and management of cemeteries, in a practical and concise form. It is hoped that cemetery officials will find the volume useful in disseminating modern ideas on the subject and in educating lot owners as to modern conditions in order that many present-day cemeteries may be improved. The Chapter headings are: Cemetery History and Burial Customs in America; The Organization and Ownership of Cemeteries; The Proper Location for a Cemetery; The Cemetery Plan; General Construction Work; Road Construction; Landscape Development; The Superintendent and His Duties; Mausoleums, Monuments, and Headstones; Rules and Regulations; Perpetual Care Fund; Cemetery Records; The Cemetery Buildings; Charges for Lot and General Services; The Improvement of Old Cemeteries; Cremation; Cemetery Law; Cemetery Literature and the A. A. C. S.

CYANIDE PRACTICE IN MEXICO.

By Ferdinand McCann. Cloth, 9¼ x 6 in., illus., 194 pp. San Francisco, The Mining and Scientific Press; London, The Mining Magazine, 1912. $2.00.

The preface states that this volume consists largely of extracts from a Spanish book by the author entitled "Beneficio de Metales de Plata y Oro por Cianuracion." These extracts have been revised, and new material has been added, it is stated, in order to embody the latest practice in various Mexican mills. The book contains, it is said, an accurate detailed description of the equipment and practice at all the important cyanide plants in Mexico, and as Mexican practice deals especially with the cyanidation of silver ores, the author hopes that the volume will be an acceptable addition to the literature on that subject. Many illustrations are given, including the flow sheets of many representative mills. The Contents are: Historical Outline of the Cyanide Process; Various Systems of Treatment; Cyanide Practice of the El Oro Mining & Railway Co., El Oro, Mexico; The Dos Estrellas Co., Tlalpujahua, Michoacan; The Mexico Mines of El Oro, Ltd., El Oro, Mexico; The Esperanza Mining Co., El Oro, Mexico; The Guanajuato Consolidated Mining & Milling Co., Guanajuato, Mexico; The Real del Monte y Pachuca Co., Pachuca, Hidalgo; In the Tailing Plant of the Blaisdell Coscotitlan Syndicate, Pachuca, Hidalgo; In the San Francisco Mill No. 1 of the Compañía Beneficiadora de Metales, Pachuca, Hidalgo; Of the San Rafael y Anexas Co., Pachuca, Hidalgo; At the Plant of the Lucky Tiger Combination Gold Mining Co., El Tigre Sonora, by D. L. H. Forbes; At the Veta Colorado, Parrel, Chihuahua, by Bernard McDonald; Cyanide Practice in Small Mills; Continuous Cyanide Treatment in Connection with Pachuca Tank; Cyanidation in Pan-Amalgamation Mills Without Change in the Machinery; Precipitation on Metallic Zinc; Treatment of Cyanide Precipitates; Index.

THE MINING WORLD INDEX

Of Current Literature, Vol. II, July-December, 1912. By George E. Sisley. Cloth, 9¼ x 6 in., 24 + 234 pp. Chicago, Mining World Company, 1913. $2.00. (Donated by *Mining and Engineering World.*)

The first volume of this Index was issued in August, 1912, and covered the first six months of that year. As stated in the title, this, the second volume, includes the period from July to December, 1912, and covers the field of the world's current literature of mining, metallurgy, and the allied industries. The subject-matter is a revise of the Index of the same subjects published weekly in the *Mining and Engineering World*, and is stated to have been issued in book form for the benefit of engineers and others who wish to keep in touch with the progress made in mining, metallurgy, etc. Some changes have been made, it is said, in the arrangement of the present volume with a view to making reference easier, the entries being alphabetical by author under the subject and including author, title of article (titles in foreign languages are usually followed by a translation or explanation in English), a brief amplification or explanation when the title is insufficient, the journal in which the article appeared, with the date and page number, the approximate number of words in the article, and the price, which arrangement, it is hoped, will be of benefit to those whose library facilities are limited. The Contents are: Part I, Ores and Mineral Products; Metals and Metal Ores; Non-Metals. Part II, Technology: Mines and Mining; Mill and Milling; Metallurgy and Chemistry; Power and Machinery. Part III, Miscellaneous.

ELECTRIC LIGHTING AND MISCELLANEOUS APPLICATIONS OF ELECTRICITY.

A Text Book for Technical Schools and Colleges. By William Suddards Franklin. Half Roan, 8¾ x 5¾ in., illus., 7 + 299 pp. New York, The Macmillan Company; London, Macmillan & Co., Ltd., 1912. $2.50.

As stated in the secondary title, this book which is a companion volume to "Dynamos and Motors," is intended as a textbook and as an elementary treatise on electrical engineering. As such, little attention, it is stated, has been given to the principles of design, the subject-matter being devoted to the purely physical problems of operating engineering. The Contents are: Installation and Operation Costs; Electric Distribution and Wiring; Alternating-Current Lines; Photometry; Electric Lamps, Lamp Shades and Reflectors; Interior Illumination; Street Illumination; Electrolysis and Batteries; Telegraph and Telephone; Appendix A, Dielectric Stresses; Appendix B, Problems; Index.

OBED HUSSEY,

Who, of All Inventors, Made Bread Cheap. Edited by Follett L. Greeno. Cloth, $7\frac{3}{4}$ x $5\frac{1}{4}$ in., illus., 228 pp. Rochester, N. Y., Follett L. Greeno, 1912.

On the title-page of this book, it is stated that it is "a true record of his [Mr. Hussey's] life and struggles to introduce his greatest invention, the reaper, and its success, as gathered from pamphlets published heretofore by some of his friends and associates, and reprinted in this volume together with some additional facts and testimonials from other sources."

SWEET'S CATALOGUE OF BUILDING CONSTRUCTION.

Architects' and Builders' Edition for 1913, Indexed by Firms Represented, by Products, and by Location; Together with a Checking List for Use in the Making of Specifications and Estimates, by William Brokaw Bamford, M. Am. Soc. C. E. Devised, Compiled, Edited and Published Annually by The Architectural Record Co. Cloth, $13\frac{1}{2}$ x 10 in., illus., 71 + 1 901 pp. New York and Chicago, Architectural Record Co., 1913.

The General Index to this volume is divided into three parts: Part I, Index to Firms Catalogued, which contains an alphabetical list of all firms included in the Catalogue; Part II, Products Index, which contains a list of all the products included in the Catalogue, arranged alphabetically, with cross-references; and Part III, Geographical Index, which contains a list of all the firms catalogued, arranged alphabetically by cities, including each firm's main and branch offices and agencies.

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BY PURCHASE

Annales des Mines ou Recueil de Memoires sur l'Exploitation des Mines et sur les Sciences et les Arts qui s'y Rattachent. Publies sous l'Autorisation du Ministre des Travaux Publics des Postes et des Telegraphes. H. Dunod et E. Pinat, Paris, 1907.

Ölmotoren in Viertakt- und Zweitaktbauart: Entwurf, Berechnung und Bau der Leichtölmotoren, Glühkopfmotoren, Hochdruckmotoren (Diesel u. a.) ferner Motorlokomobilen, Schiffs- und Automobilmotoren, Motorlokomotiven, Triebwagen, Luftschiffsmotoren. Von H. Haeder. 2 Vol. Otto Haeder, Wiesbaden, 1912.

Coal: Its Composition, Analysis, Utilization and Valuation. By E. E. Somermeir. McGraw-Hill Book Co., New York and London, 1912.

The American Year Book: A Record of Events and Progress. 1912. Edited by Francis G. Wickware Under Direction of a Supervisory Board Representing National Learned Societies. D. Appleton and Co., New York and London, 1913.

The New City Government, A Discussion of Municipal Administration Based on a Survey of Ten Commission Governed Cities. By Henry Bruère and William Sheperdson, Coöperating in the Collection and Tabulation of Material Respecting Commission Government. D. Appleton and Co., New York and London, 1912.

German Varnish-Making. By Max Bottler. Authorized Translation with Notes on American Varnish and Paint Manufacture, by Alvah Horton Sabin. John Wiley & Sons, New York and London, 1912.

The Graphic Method of Influence Lines for Bridge and Roof Computations. By William H. Burr and Myron S. Falk. Third Edition. John Wiley & Sons, New York and London, 1912.

The Design and Construction of Steam Turbines, A Manual for the Engineer. By Harold Medway Martin. Longmans, Green and Co., New York and London, 1913.

Methods of Measuring Electrical Resistance. By Edwin F. Northrup. McGraw-Hill Book Co., New York and London, 1912.

Motor Car Construction: A Practical Manual for Engineers, Students, and Motor Car Owners, with Notes on Wind Resistance and Body Design. By Robert W. Brewer. D. Van Nostrand Company, New York; Crosby Lockwood and Son, London, 1912.

Concentrating Ores by Flotation, Being a Description and History of a Recent Metallurgical Development, Together with a Summary of Patents and Litigation. By Theodore J. Hoover. Mining and Scientific Press, San Francisco; The Mining Magazine, London, E. C., 1912.

Der Eisenbau: Ein Hilfsbuch für den Brückenbauer und Eisenkonstrukteur. Von Luigi Vianello. In zweiter Auflage umgearbeitet und erweitert von Carl Stumpf. R. Oldenbourg, München und Berlin, 1912.

Die Eisenbetonliteratur bis Ende 1910: 1, Inhalts-Verzeichnis der Zeitschrift Beton u. Eisen 1901 bis 1909; 2, Schlagwortverzeichnis zum Handbuch für Eisenbetonbau; 3, Zeitschriftenschau der gesamten Eisenbetonliteratur bis Ende 1910. By Richard Hoffmann. Wilhelm Ernst & Sohn, Berlin, 1911.

Forscherarbeiten auf dem Gebiete des Eisenbetons: Beitrag zur Berechnung der kreuzweise bewehrten Eisenbetonplatten und deren Aufnahmeträger. Von Arturo Danusso. Heft 21. Wilhelm Ernst & Sohn, Berlin, 1913.

Handbuch der Ingenieurwissenschaften: Fünfter Teil. Der Eisenbahnbau; Betriebseinrichtungen. Von S. Scheibner, F. Loewe und H. Zimmermann. Sechster Band. 3 Vol. Wilhelm Engelman, Leipzig, 1913.

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GRAY, HARRY MATT. Res. Engr., Hazen & Whipple, 21 Balding Ave., Poughkeepsie, N. Y.		Mar. 4, 1913
GREELEY, SAMUEL ARNOLD. Asst. Engr., Sewage Disposal Div., San. Dist. of Chicago, Winnetka, Ill.	Jun. Assoc. M.	Feb. 5, 1907 Mar. 4, 1913

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
HARMON, IVAN GUY. Care, Mercantile Trust Co., St. Louis, Mo		Dec. 3, 1912
HEYMAN, WILLIAM. 325 Montgomery St., Jersey City, N. J	Jun. Assoc. M.	June 30, 1911 Mar. 4, 1913
HOWARD, JOHN WARDWELL. Instr. in Civ. Eng., Mass. Inst. of Tech., Boston, Mass		Mar. 4, 1913
KESNER, HENRY JAMES. Asst. Prof. of Civ. Eng., Univ. of California, Civ. Eng. Bldg., Berkeley, Cal	Jun. Assoc. M.	Dec. 6, 1910 Mar. 4, 1913
KINGSLEY, CHARLES BROWN. 82 King St., East, Toronto, Ont., Canada		Mar. 4, 1913
LOGAN, HAL HELM. 504 Empire Bldg., Denver, Colo		Mar. 4, 1913
MACOMBER, STANLEY. City Engr., City Hall, Centralia, Wash		Mar. 4, 1913
MCCRORY, THOMAS GEORGE. Engr.-in-Chg., International Contract Co., 501 Central Bldg., Seattle, Wash	Jun. Assoc. M.	Dec. 1, 1908 Mar. 4, 1913
MAHON, JOHN MONTGOMERY, JR. Asst. Engr., Pennsylvania Dept. of Health, 219 Walnut St., Harrisburg, Pa		Mar. 4, 1913
MIETH, RICHARD ELAM. Mgr., Portland Bridge Co., 1125 Yeon Bldg., Portland, Ore	Jun. Assoc. M.	May 1, 1906 Mar. 4, 1913
OKES, DAY IRA. Chf. Engr., The Kettle River Co., 2700 Pillsbury Ave., Minneapolis, Minn	Jun. Assoc. M.	Mar. 1, 1910 Mar. 4, 1913
PAINE, PAUL MCCLARY. Engr. and Asst. Supt., Honolulu Consolidated Oil Co., Fellows, Cal		Mar. 4, 1913
PANI, ARTURO. Chf. of Dept. of Public Works, Govt. Ministry of Communications and Public Works, 5a del Fresno No. 184, City of Mexico, Mexico		Mar. 4, 1913
SANGER, WALTER MAX. Engr., George M. Jones Coal Co., 635 Lincoln Ave., Toledo, Ohio	Jun. Assoc. M.	May 5, 1908 Mar. 4, 1913
SEIBERT, PERCY ALLEN. Mgr. and Legal Representative, Andes Tin Co., Casilla No. 378, La Paz, Bolivia		Jan. 7, 1913
SMITH, CLARENCE URLING. Res. Engr., C., M. & St. P. Ry., P. O. Box 23, Chanhassen, Minn	Jun. Assoc. M.	Sept. 5, 1911 Feb. 4, 1913
SMITH, HAROLD GARFIELD. Room 605, Deseret News Bldg., Salt Lake City, Utah		Feb. 4, 1913
STEVENSON, JAMES CUMMIN. Supt. of Constr., H. L. Stevens & Co., Main Ave. and Houston St., San Antonio, Tex		Mar. 4, 1913
WHITHAM, PAUL PAGE. Chf. Engr., Port of Seattle, 843 Central Bldg., Seattle, Wash		Oct. 1, 1912
YOUNG, OLIVER EARLE. Care, Amalgamated Phosphate Co., Chicora, Fla	Jun. Assoc. M.	June 6, 1911 Mar. 4, 1913

JUNIORS

	Date of Membership.
BATES, CLARENCE MYERS. Res. Engr., Waddell & Harrington, Cor. 11th St. and St. Paul Ave., Tacoma, Wash.	Feb. 4, 1913
BROWN, EARL DANIEL. 2164 Forty-second Ave., Oakland, Cal	Oct. 1, 1912
DRURY, WILLIAM FISHER. Engr.-in-Chg., Veazie Dam, for Bangor Power Co., Box 261, Orono, Me	Mar. 4, 1913
EURICH, RICHARD HENDERSON. 144 Union St., Montclair, N. J	Mar. 4, 1913
GREENE, ROBERT NESBITT. Engr. for S. H. McKenzie, Box 375, Southington, Conn	Mar. 4, 1913
GUILLEMETTE, JOSEPH DYDEME. 434 Rebecca Ave., Wilkinsburg, Pa	Oct. 29, 1912
HOWARD, CECIL WARD. Care, Wells Bros. Constr. Co., Cincinnati, Ohio	Jan. 7, 1913
PARTRIDGE, JOHN FREDERICK. Escalon, Cal	Feb. 4, 1913
SCHROEDER, SEATON, JR. Care, Barclay Parsons & Klapp, Altmar, N. Y	Mar. 4, 1913
SHIELDS, JAMES RALPH. Asst. in Testing Laboratory, Univ. of California, 2119 McKinley Ave., Berkeley, Cal	Oct. 1, 1912
STAVA, WILLIAM. Box 480, Escalon, Cal	Feb. 4, 1913
STEINHAUSER, HARRY HERMAN. Asst. to Engr.-in-Chg. of Hydraulics, Rochester Ry. & Light Co., 34 Clinton Ave., N., Rochester, N. Y	Feb. 4, 1913
TATE, ROBERT L'HOMMEDIEU. Care, Hydro-Elec. Co. of W. Va., Bruceton Mills, W. Va	Mar. 4, 1913

CHANGES OF ADDRESS

MEMBERS

ANDROS, FREDERIC WILLIAM. 718 Calle Agustinas, Santiago, Chili.

ARTHUR, HOWARD ELMER. Stearns, N. C.

ATWOOD, THOMAS CLARK. Div. Engr., Dept. of Water Supply, Gas, and Electricity, 13 Park Row, New York City (Res., 8 Amackassin Terrace, Yonkers, N. Y.).

AUCHINCLOSS, WILLIAM S. Atlantic Highlands, N. J.

BLAND, JOHN CARLISLE. Engr. of Bridges, Penn. Lines West of Pitts., Room 1115, Pennsylvania Station, Pittsburgh, Pa.

BONSTOW, THOMAS LACEY. With S. Pearson & Son, Sucesores, S. A., Apartado 68, Puerto Mexico, Ver., Mexico.

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BURROWES, HARRY GILBERT. Cons. Engr., Pittsville, Pa.

BYERS, MAXWELL CUNNINGHAM. Asst. to the Pres., G. N. Ry., 655 Goodrich Ave., St. Paul, Minn.

MEMBERS (*Continued*)

COLE, WILLIAM WEEDEN. Mgr. of Utilities Dept., Dodge & Day, 611 Chestnut St., Philadelphia, Pa.

ERNST, OSWALD HERBERT. Brig.-Gen., U. S. A. (*Retired*); Chairman, Am. Section, International Waterways Comm., 1321 Connecticut Ave., Washington, D. C.

ESSELSTYN, HORACE HOVEY. Constr. Engr., The Edison Illuminating Co., 67 Kenilworth Ave., Detroit, Mich.

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FOX, JOHN ANGELL. Secy.-Mgr., Mississippi River Levee Assoc., 416 Scimitar Bldg., Memphis, Tenn.

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HERBERT, HARRY MONMOUTH. Cons. Engr., Bound Brook, N. J.

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LAUB, HERMANN. Cons. Engr., 611 Keystone Bldg., Pittsburgh, Pa.

MACLAY, EDGAR GLEIM. (Central Contr. Co.), 421 Beatty Bldg., Houston, Tex.

McGONIGLE, CHARLES JOSEPH. Cons. Structural Engr., 815 Chamber of Commerce Bldg., Portland, Ore.

MATHESON, ERNEST GEORGE. Coquitlam, B. C., Canada.

MOLITOR, DAVID ALBERT. Care, Toronto Harbor Commrs., 76 Adelaide St., West, Toronto, Ont., Canada.

MOODY, BURDETT. Chf. Engr., Water Dept., 72 North Fair Oaks Ave., Pasadena, Cal.

NICHOLS, EDWIN JAY. Asst. Engr., St. Louis, Southwestern Ry., 424 West Ferguson St., Tyler, Tex.

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PHILLIPS, ARTHUR LOUIS. Asst. Div. Engr., Tela R. R., Tela, Honduras.

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MEMBERS (*Continued*)

STEIN, CHARLES HENRY. Engr., M. of W., C. R. R. of N. J., 69 Danforth Ave., Jersey City, N. J.

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STUBBS, LINTON WADDELL. 657 Herndon Ave., Shreveport, La.

SWINDLEHURST, HAROLD LIONEL. Shire Engr., Goulburn City, New South Wales, Australia.

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HOOD, JOSEPH NELSON. Care, Davis & Davis, 14 St. Peter St., Quebec, Que., Canada.

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HUNTING, EUGENE NATHAN. Archt. and Engr. (The Hunting Davis Co.), Century Bldg., Pittsburgh, Pa.

JOHNSON, RANKIN. Huntington, N. Y.

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MOODY, JOSEPH ELBERT. With Hurley-Mason Co., Engrs. and Contrs., Sherwood Bldg., Spokane, Wash.

MURRAY, CLARE DELOSS. Care, Utah Power & Light Co., Grace, Idaho.

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OAKLEY, GEORGE ISRAEL. Little Falls, N. Y.

ASSOCIATE MEMBERS (*Continued*)

OSBOURN, HENRY VAN BUREN. Insp., Bureau of Highways (Res., 805 East Washington Lane), Philadelphia, Pa.

PENDLETON, DAVID ELLIOTT. Room 67 Dearborn Station, Chicago, Ill.

ROACH, JAMES HOWARD. Asst. Engr. of Constr., The L. S. & M. S. Ry., 11512 Ashbury Ave., Cleveland, Ohio.

RUCKES, JOSEPH JOHN, JR. Engr., Barrett Mfg. Co., 954 Anderson Ave., New York City.

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SCHMID, FRANCIS RAUCH. Asst. Engr., N. Y. C. & H. R. R. R., 540 Lexington Ave., Room 622, New York City.

SHAW, GEORGE HERBERT. San. Engr., Bureau of Animal Industry, Dept. of Agri., Washington, D. C.

SINNICKSON, GEORGE ROSENGARTEN. Care, W. J. & S. R. R., Camden, N. J.

SKINNER, FREDERICK GARDINER. 41 Willis Ave., West, Detroit, Mich.

STANAGE, JOHN LYNCH. Supt. of Constr., Roach & Manigan, Room 411 Times-Herald Bldg., Waco, Tex.

STEFFENS, WILLIAM FREDERICK. Asst. Chf. Engr., C. & O. R. R., Richmond, Va.

STEHLE, FELIX CHARLES. Towanda, Pa.

STRICKLER, FREDERICK WINEMAN. Care, Erie R. R., Cambridge Springs, Pa.

TARR, CHARLES WINTHROP. 12 East Haverhill St., Lawrence, Mass.

TEFFT, WILLIAM WOLCOTT. Civ. and Hydr. Engr., 708 Wildwood Ave., Jackson, Mich.

THOMAS, WILLIAM EDWARD. 68 East 86th St., New York City.

TOMLINSON, CARL PERKINS. Asst. Supt. of Constr., Stone & Webster Eng. Corporation, 38 Morris St., Danbury, Conn.

VOORHES, KIMBROUGH ENOCH. 41 North Arlington Ave., East Orange, N. J.

WHITE, ARTHUR BURR. Civ. and Hydr. Engr. (Bixby & White), 105 Henne Bldg., Los Angeles, Cal.

ASSOCIATES

MARSH, ALBERT LEREAUX. 282 Montclair Ave., Newark, N. J.

OLDS, WILLIAM CLARENCE. 1817 F St., N. W., Washington, D. C.

PARSHALL, RALPH LEROY. Care, U. S. Dept. of Agri., Colorado Experiment Stations, Fort Collins, Colo.

JUNIORS

ARMSTRONG, GEORGE SIMPSON, JR. Care, Suffern & Son, 21 Bold St., Hamilton, Ont., Canada.

BEALL, PENDLETON. P. O. Box 72, Poughkeepsie, N. Y.

BOLTON, FRANK LEONARD. Designing and Const. Engr. (Bolton, Ruetenik & May), Bend, Ore.

BOWMAN, RALPH MCLANE. Reinforced Concrete Designer, Corrugated Bar Co., Room 402, Mutual Life Bldg., Buffalo, N. Y.

BROWN, CLAUDE OSGOOD. 30 Columbia Park, Haverhill, Mass.

BURNHAM, GEORGE EARLE. Res. Engr., Manila R. R., Manila, Philippine Islands.

JUNIORS (*Continued*)

Burrowes, Robert William. P. O. Box 556, Palm Beach, Fla.
Carpenter, J. C. Dist. Engr., Albay, Albay, Philippine Islands.
Cummin, Hart. Care, J. G. White Eng. Corporation, 43 Exchange Pl., New York City.
Davis, Harold Martin. With Sanford E. Thompson, Newton Highlands, Mass.
DuBois, George Bache. Care, Myers & McWilliams, Pittsford, N. Y.
Dubois, Gustavo Adolfo. Calle 17, No. 10, esg. á M, Havana, Cuba.
Field, Clesson Herbert. With Lackawanna Steel Co. (Res., 516 Elmwood Ave.), Buffalo, N. Y.
Hadley, Homer More. Res. Engr., C. N. P. Ry., Strawberry Vale, Vancouver Island, B. C., Canada.
Hawley, Charles Burridge. 1025 First St., Jackson, Mich.
Hughes, William Richard, Jr. 246 West Forest Ave., Detroit, Mich.
McClure, Hunter. 436 Madison St., Gary, Ind.
Malmros, Nils Lorentz Alfred. Supt., Ernest Flagg, 136 East 4th St., Cincinnati, Ohio.
Malony, Walden Le Roy. Asst. Supt., Bates & Rogers Constr. Co., P. O. Box 1078, Spokane, Wash.
Moore, Walter Smyth. Asst. Engr., L. & N. R. R., Versailles, Ky.
Nevius, Searle Brown. Structural Steel Draftsman, Galloway & Markwart, 2294 Filbert St., San Francisco, Cal.
Powell, William Jenner. Office Engr., City Engr.'s Office, Dallas, Tex.
Russell, Alexander Allen MacVicar. 2712 Stuart St., Berkeley, Cal.
Shaw, Guy Ray. 225 Fifth St., Room 807, Des Moines, Iowa.
Silsbee, James Alfred. Elmira, N. Y.
Sleppy, Kirby Baldwin. Care, Inter-State Telegraph Co., Bishop, Cal.
Snyder, Hubert Earl. Manbar, W. Va.
Stewart, Charles Sumner. North Platte, Nebr.
Stirling, Vincent Reynolds. Chf. Engr., Govt., Moro Province, Zamboanga, Moro Province, Philippine Islands.

REINSTATEMENTS

MEMBERS	Date of Reinstatement.
McClintock, William Edward	Mar. 4, 1913
ASSOCIATE MEMBERS	
Downman, Julian Romney	Mar. 4, 1913

RESIGNATIONS

MEMBERS	Date of Resignation.
Phillips, Joseph Leslie	Mar. 4, 1913
ASSOCIATE MEMBERS	
Huntington, George Danforth	Mar. 4, 1913
Ravenscroft, Edward Hawks	Mar. 4, 1913
Whitney, Harrison Allen	Mar. 4, 1913

DEATHS

ARCHIBALD, PETER SUTHER. Elected Member, January 7th, 1885; died March 16th, 1913.

COOMBS, PHILIP HENRY. Elected Member, March 7th, 1906; died March 6th, 1913.

CRYSLER, ARTHUR GARFIELD. Elected Associate Member, August 31st, 1909; died October 22d, 1912.

ELLIOTT, JOHN STUART. Elected Associate Member, April 6th, 1892; died March 25th, 1913.

KELLOGG, CHARLES. Elected Member, June 2d, 1880; died March 23d, 1913.

KUERSTEINER, EMIL EDWARD. Elected Member, December 1st, 1897; died March 10th, 1913.

SPEARMAN, CHARLES. Elected Associate Member, October 5th, 1909; died August, 1912.

Total Membership of the Society, April 3d, 1913,
6 858.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(March 5th to April 1st, 1913)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Institute Indicator*, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *Engineering Record*, New York City, 10c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Elektrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer*, Scranton, Pa., 25c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $4.

(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England, 6d.
(69) *Der Eisenbau*, Leipzig, Germany.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Mining and Engineering World*, Chicago, Ill., 10c.
(83) *Gas Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga., 10c.

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Troubles Resulting from Proximity to Gas and Oil Wells. L. M. Jones. (13) Mar. 6.
Gas and Oil Wells in Coal Fields. George S. Rice. (13) Mar. 6; (82) Mar. 22.
Suggestion for Laws and Regulations (Location of Oil-Wells). (13) Mar. 6.
Electric Cables for Shafts of Mines.* E. Kilburn Scott, A. M. Inst. C. E. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Serial beginning Mar. 7.
Mining Operations in Idaho During 1912.* Robert N. Bell. (82) Mar. 8.
Proposed Regulation of Gold-Dredging. Charles Janin. (103) Mar. 8.
Shaft Sinking at the Indiana Mine.* Claude T. Rice. (16) Mar. 8.
Notes on Mine Sampling.* G. C. Bateman. (Paper read before the Canadian Min. Inst.) (16) Mar. 8.
Bucket Dredger for Nigerian Tin Deposits.* (11) Mar. 14.
Approved Safety Lamps.* (57) Mar. 14.
Electric Equipment, American Nettie Mine. T. A. Tefft. (16) Mar. 15.
Sulphur and Iron Deposits of Virginia.* J. F. Springer. (82) Mar. 15.
The Use of Coal-Cutters for Blackband Ironstone at Parkhouse Mine.* (22) Mar. 21.
Iron Ore Sorting Plant at the Gellivare Mines, Sweden.* Harry J. H. Nathorst. (From *Jern-Kontorets Annaler.*) (22) Mar. 21.
Methods and Costs, Mother Lode Mine, B. C.* E. Hibbert. (16) Mar. 22.
Graphics Applied to Fault Problems.* (Mining.) E. R. Rice. (16) Mar. 22.
Progressive Mines in the Iron River District.* Geo. E. Edwards. (82) Mar. 22.
The Monarch Mine in British Columbia.* Newton W. Emmens. (82) Mar. 22.
Rock Asphalt Deposits of Oklahoma.* L. C. Snider. (82) Mar. 22.
Acetylene Lamps in Mines.* R. Cremer. (68) Serial beginning Mar. 22
A Tungsten Mine in Nova Scotia.* V. G. Hills. (103) Mar. 22.
Exhaust Steam and Its Utilization at Collieries and Mines.* J. M. Gordon. (Paper read before the Canadian Min. Inst.) (96) Mar. 27.
A Mine Locomotive Designed for Accessibility.* (13) Mar. 27.
Lowering Supplies at Western Mines.* Claude T. Rice. (82) Mar. 29.
Electric *vs.* Compressed Air Hoists.* (For Mines.) K. A. Pauly. (16) Mar. 29.
Reconstruction of an American River Flume.* Lewis H. Eddy. (16) Mar. 29.
Seilbahn für Vergnügungsreisende im Kgl. Salzbergwerk zu Berchtesgaden.* L. Schütt. (48) Jan. 11.

Miscellaneous.

Recent Progress in Applied Chemistry and in Engineering. James O. Handy. (58) Feb.
Design of Retaining Walls. Alfred W. Hoffmann. (87) Serial beginning Mar.
On the Physics of the Atmosphere.* W. J. Humphreys. (3) Mar.
The Engineer in the Building of the Republic. Isham Randolph. (3) Mar.
The Development of Research Work in Timber and Forest Products. E. Russell Burdon. (29) Mar. 7.
Air Currents and Their Relation to the Acoustical Properties of Auditoriums, with Application of the Conclusions to Ventilating Systems.* F. R. Watson. (14) Mar. 8.
Oil Pipe Lines in California.* B. K. Stroud. (13) Mar. 13.

Municipal.

The Surface Dressing of Roads, the Tarflux Process.* (104) Feb. 28.
Paving in Salt Lake City.* D. H. Blossom. (Paper read before the Utah Soc. of Engrs.) (1) Mar.
European Practice in Testing Road Materials. M. A. Mesnager. (Paper read before the Inter. Assoc. for Testing Materials.) (67) Mar.
Paving in Trenton, N. J., Under the New Commission.* Harry F. Harris. (60) Mar.
Creosoted Wood Block Pavement in New York.* (60) Mar.
Baltimore the City of Parks.* Stuart Stevens Scott. (60) Mar.
Municipal Asphalt Plants; Cost of Operation and Estimates for the Establishment of a Plant for the District of Columbia. David E. McComb. (Report to the Commrs. of the District of Columbia.) (86) Mar. 5.

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Municipal—(Continued).

A Footway Tunnel in New York City. (13) Mar. 6.
Some Features of Macadam Construction. T. R. Agg. (Paper read before the Am. Road Builders' Assoc.) (96) Mar. 6.
Plant Equipment. F. E. Ellis. (Paper read before the Am. Road Builders' Assoc.) (96) Mar. 6.
Riverbank Experimental Road.* (14) Mar. 8.
Concrete Road Construction in Milwaukee County, Wis., and in Winona County, Minn. H. J. Knelling and O. B. Leland. (From papers read before the Minnesota and Wisconsin Soc. of Engrs.) (86) Mar. 12.
Effect of Heavy Motor Traffic on Pavements. (86) Mar. 12.
Repair and Maintenance of Roads. L. I. Hewes. (Paper read before the Ontario Good Roads Assoc.) (96) Mar. 13.
Earth and Gravel Roads. Robert C. Terrell. (Paper read before the Am. Road Builders' Assoc.) (96) Mar. 13.
Recent Motor Equipment in the New York Fire Department.* (11) Serial beginning Mar. 14.
Methods and Costs of Constructing Three Types of Paving for Street Railway Tracks.* D. B. Davis. (86) Mar. 19.
Some Costs on the Construction of Concrete Pavement. Carl M. Boynton. (Paper read before the Am. Soc. of Eng. Contractors.) (86) Mar. 19.
Wood Paving with Lug-Blocks.* (13) Mar. 20.
Good Roads in Ontario. W. A. McLean. (Paper read before the Ontario Good Roads Assoc.) (96) Mar. 20.
Road Construction. A. McLean. (96) Mar. 20.
Some Municipal Works and Practice in Leeds. W. T. Lancashire. (Paper read before the Inst. of Min. and County Engrs.) (104) Mar. 21.
Karachi.* J. Forrest Brunton. (29) Mar. 21.
A Classification of Road Building Rocks. Charles P. Berkey. (Paper read before the Am. Soc. for the Advancement of Science.) (86) Mar. 26.
Building a Paved Roadway Across a Swamp. James Owen. (Abstract of paper read before the County Engrs. of New Jersey.) (13) Mar. 27.
A Comparison of Recent Bids for Various Types of Paving in the New York Navy Yard. Walter H. Allen. (13) Mar. 27.
City Pavements. G. G. Powell. (Abstract of paper read before the Ontario Good Roads Assoc.) (96) Mar. 27.
Recommendations for Broad Street Paving, Newark. (14) Mar. 29.
Standardizing Highway Construction.* Charles E. Foote. (46) Mar. 29.
Central Purchase and Distribution of Supplies for New York City, Plan for Controlling and Standardizing Purchasing Methods of 128 Different Municipal Departments and Boards. (14) Mar. 29.
Zur Geschichte der Ziegelstrasse in Berlin. Ernst Friedel. (80) Serial beginning Dec. 3.

Railroads.

Rolling-Stock on the Principal Irish Narrow-Gauge Railways.* R. M. Livesey. (75) July.
Hump *vs.* Flat Shunting. (Abstract from *North Eastern Railway Magazine.*) (23) Jan. 3.
New Garratt Locomotives, Tasmanian Government Railways.* (23) Jan. 3.
The Relaying and Improvement of the Berks & Hants Junction, Reading, Great Western Railway.* (23) Jan. 17.
New Great Central Locomotive *Sir Sam Fay* to be Exhibited at the Ghent Exhibition.* (23) Jan. 17.
Tables for Finding Proper Tonnage Rating; Acceleration and Retardation Figures, and Their Use in Solving Problems of Railway Location and Train Loading.* Paul M. La Bach. (23) Jan. 17.
Rapid Acting Vacuum Brake Trials in Austria.* (23) Jan. 17.
Electric Baggage Trucks.* (23) Jan. 24.
Electrification of Heavy Grades, This May be Found Less Expensive than Grade Reduction.* C. L. de Muralt. (23) Jan. 31.
Heavy Tank Locomotive, Londonderry & Lough Swilly Railway.* (23) Jan. 31.
Freight Train Control, North-Eastern Railway; Middlesbrough Division.* (23) Feb. 7; (22) Feb. 28.
The "Safety First" Movement. Geo. Bradshaw. (61) Feb. 18.
35-Ton Bogie Rail Wagons, Caledonian Railway.* (23) Feb. 21.
Two European Single-Phase Railways: The Mittenwald Electric Railway and Rjukan Railway.* (26) Feb. 28.
Circulation of Water in Locomotive Boilers.* George L. Fowler. (47) Feb. 28.
Six-Coupled Bogie Express Locomotive for the Great Central Railway.* (11) Feb. 28.
Railroad Accidents; Their Causes and Remedy. D. F. Jurgensen. (Paper read before the Civ. Engrs.' Soc. of St. Paul.) (1) Mar.

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Railroads (**Continued**).

An Unusual Retaining Wall of Reinforced Concrete.* Albert J. Himes, M. Am. Soc. C. E. (36) Mar.
Rail Creeping.* J. G. Van Zandt. (87) Mar.
Safety on Railroads. J. W. Coon. (65) Mar.
Centralia Terminal, I. C. R. R.* (87) Mar.
Union Station, Fort Smith, Ark.* (87) Mar.
Concrete Practice No. 6, Kansas City Southern Ry. Co.* A. M. Wolf. (87) Mar.
Snow Hill Station, Great Western Railway.* R. P. Mears, Assoc. M. Inst. C. E. (Abstract of paper read before the Junior Inst. of Engrs.) (21) Mar.; (23) Jan. 24.
Simplex Concrete Piles at Alost Station, Belgium.* E. Creplet. (From *Bulletin Technique* du Cercle des Chefs de Section des Chemin de Fer de l'Etat.) (21) Mar.
Long Switches for Fast Running; Chemins de Fer du Nord.* (21) Mar.
Automatic Block Signals on the Atlantic Coast Line R. R.* B. W. Meisel. (87) Mar.
Assigning Cause of Failure in Rail Failure Reports. P. M. La Bach. (87) Mar.
Canadian Pacific 4-6-2 Type Locomotive.* W. H. Winterrowd. (25) Mar.
Locomotive Deck Shields.* Walter R. Hedeman. (25) Mar.
Experimental Electric Locomotives in France. (25) Mar.
Pittsburgh & Lake Erie Two Car Gas-Electric Car.* (25) Mar.; (18) Mar. 8.
Locating Defective Car Wheels.* D. C. Buell. (25) Mar.
The Gee Locomotive Stoker.* (25) Mar.; (15) Mar. 14.
The Laying of Rails on Wooden Sleepers for Busy Express Lines.* C. W. Van Dÿk. (Paper read before the Royal Institute of Dutch Engrs.; from *De Ingenieur.*) (88) Mar.
Locomotive Development on the Prussian-Hessian State Railway and Some Results Obtained in Practice with Superheated-Steam Locomotives. E. Meyer. (From *Annalen für Gewerbe und Bauwesen.*) (88) Mar.
Hardwood Pads for Railway Sleepers.* M. Matthael. (From *Annalen für Gewerbe und Bauwesen.*) (88) Mar.
The Karns Tunneling Machine.* O. J. Grimes. (13) Mar. 6.
A Railway Grade Crossing with Steel Foundation.* (13) Mar. 6.
Train Control on the Rhymney Railway.* (23) Mar. 7.
Narrow Gauge Locomotive for the Rhodesian Railways.* (12) Mar. 7.
Grand Trunk Terminal at Ottawa, Ont.* (15) Mar. 7.
Fuel Oil Installations, Northern Pacific Ry.* (18) Mar. 8.
Vital Factors in Car Economy. Chas. A. Lindstrom. (Abstract of paper read before the New England R. R. Club.) (18) Mar. 8.
Scaling Mountain Peaks by Elevator.* (19) Mar. 8.
Electric Railway in the French Pyrenees.* (27) Mar. 8.
A New Single-Phase Railway in Norway. (12) Mar. 14.
The Kiangsu-Chekiang Railways.* Lewis R. Freeman. (15) Mar. 14.
Development in the Use of Screw Spikes.* (15) Mar. 14.
Third and Fourth Track Construction.* (15) Mar. 14.
Creosoting Plant near Connellsville, Pa.* (15) Mar. 14.
Rebuilding of the Chicago Clearing Yard.* (18) Mar. 15; (15) Mar. 21.
Construction Work of the Kansas City Terminal Ry.* (18) Mar. 15.
Railway Terminals.* L. C. Fritch. (Paper read before the Canadian Ry. Club.) (18) Mar. 15.
Wrecking Cars of 150 Tons' Capacity, Norfolk & Western Ry.* (18) Mar. 15. (15) Mar. 14.
Maintenance of the Pennsylvania Railroad Electric Locomotives.* (17) Mar. 15.
Experiences with Concrete Poles by the New York State Railways.* (17) Mar. 15.
Standardization on the Michigan United Traction.* (17) Mar. 15.
Road Improvements of the Northern Pacific Ry. in Washington.* (18) Mar. 15.
Double-Tracking between Calgary and Vancouver. (14) Mar. 15.
Drainage of a Railway Roadbed, Methods for Disposal of Surface and Subsurface Water. J. T. Bowser. (14) Mar. 15.
Formulas for Radial Distances Between Curves (Railroad Structures).* George Paaswell. (13) Mar. 20.
Concrete in Railroad Work. M. S. Long. (Paper read before the National Assoc. of Cement Users.) (96) Mar. 20.
Novel Tunnel Work on the Cleveland Short Line Ry.* (13) Mar. 20.
Mine Locomotives in the Dortmund District.* (57) Mar. 20.
Arabia's Only Railway. (29) Mar. 21.
Trussed-Concrete Construction in Railway Work, the Hy-Rib.* (23) Mar. 21.
Postal Car Illumination Tests.* (15) Mar. 21.
Important Improvement Work on the C., M. & St. P.* (15) Mar. 21; (18) Mar. 29.
The Washington & Old Dominion Railway.* W. N. Smith. (17) Mar. 22.
Analysis of the Determination of Economical Freight-Train Tonnages. (14) Serial beginning Mar. 22.
Baldwin Locomotives Built During 1912. (15) Mar. 28.

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Report on Glen Loch Derailment. (Abstract of Report of the Interstate Commerce Comm.) (15) Mar. 28.
Economical Limits of Grade Reduction. Walter Loring Webb. (15) Mar. 28.
Express and Freight Traffic in Providence, R. I.* (17) Mar. 29.
L'Eclairage par l'Acétylène Dissous des Voitures des Chemins de Fer du Sud de l'Autriche.* (33) Feb. 8.
Die Mallet-Lokomotive der Harzquer- und Brockenbahn. Heinrich Papst. (48) Jan. 25.
Schlafwagen III. Klasse der norwegischen Staatseisenbahnen.* P. Hoff. (48) Feb. 8.
Die Staatsbahnen Bulgariens.* Franz Manek. (53) Feb. 14.
Stossvorgang beim Auffahren eines Zuges auf einen Bremsschlitten.* F. Besser. (102) Serial beginning Feb. 15.
Neue Erfahrungen mit Eisenbetonschwellen.* R. Bastian. (78) Serial beginning Feb. 26.
Zahnrad-gegen Schubstangenmotor bei elektrischen Lokomotiven.* (41) Feb. 27.
Formänderungen am schwebenden Schienenstosse.* E. C. W. van Dyk. (102) Mar. 1.
Neuere Maschinen zum Schleifen von Achsschenkeln.* Simon. (102) Mar. 1.
Zum Gotthardvertrag. (107) Mar. 8.
Rutschung bei Hohtenn auf der Südrampe der Lötschbergbahn.* (107) Mar. 15.
Einrichtungen zur Ausbesserung der Rauch-und Ueberhitzer-Röhren der Heissdampflokomotiven.* Uhlmann. (102) Mar. 15.
Bremsschlauch-Werkstatt der Süd-Pacific-Bahn in Los Angeles.* B. Schwarze. (102) Mar. 15.
Zum Verhalten von Eisenbahnfahrzeugen in Gleisbogen.* Heumann. (102) Serial beginning Mar. 15.

Railroads, Street.

Electrification of the Berlin City Circular and Suburban Railways. (23) Serial beginning Jan. 3.
Hamburg Subway and Elevated Railway.* (17) Mar. 8.
Gasoline Surface Cars in India.* (17) Mar. 8.
Double-Deck Car for Washington.* (17) Mar. 8.
Reinforced-Concrete Trolley Poles in Cleveland.* L. P. Crecelius. (Abstract from *Aera.*) (13) Mar. 13.
Contracts Signed for New York Subways. (18) Mar. 15.
Extensions of the Boston Subway.* (18) Mar. 15.
Scientific Design of Carhouses and Shops; Methods Used in Planning an Electric Railway Maintenance Plant.* C. A. Neff and T. P. Thompson. (14) Mar. 15. (17) Mar. 15.
Track Maintenance and Reconstruction in San Francisco.* (17) Mar. 15.
Shops and Carhouses of the Hamburg Rapid Transit System.* (17) Mar. 15.
Maintenance of Electrical Equipment of Cars in Brooklyn.* (17) Mar. 15.
Fond du Lac Avenue Carhouse of the Milwaukee Electric Railway & Light Company.* (17) Mar. 15.
Novel Oil House and Oil Delivery Car for the Chicago Railways.* (17) Mar. 22.
Conveyor and Elevator Machinery for Subway Excavation. (14) Mar. 29.
Experience with One-Man-Operated Cars. (17) Mar. 29.
An Improved Type of Articulated Car.* (17) Mar. 29.
New Cars for Liverpool.* (17) Mar. 29.
The Lexington Avenue Subway Four-Track Under the Harlem River.* (46) Mar. 29.
Die städtische Untergrundbahn in Berlin-Schöneberg.* Platzmann. (41) Serial beginning Mar. 6.

Sanitation.

Street Cleaning in Downtown Chicago.* Richard T. Fox. (4) Feb.
A Typical Rural Typhoid Fever Outbreak. John H. Mahon, Jr. (98) Feb.
The National Aspect of the Reclamation of Swamp and Overflowed Lands. Edmund T. Perkins. (4) Feb.
Provision of Institutions for the Treatment of Tuberculosis.* (104) Feb. 28.
Modern Sewer Cleaning in Milwaukee, Wis.* Henry J. Kruse. (60) Mar.
Cleaning the Sewage Tank at Galva, Ill.* Lloyd Z. Jones. (60) Mar.
Hygienic Aspects of Gas. J. W. Graham. (Paper read before the Manchester and District Junior Gas Assoc.) (66) Mar. 4.
Hygienic Equipment of a High School.* (101) Serial beginning Mar. 7.
Air Currents and Their Relation to the Acoustical Properties of Auditoriums, with Application of the Conclusions to Ventilating Systems.* F. R. Watson. (14) Mar. 8.
Relation of Pure Water to Typhoid Fever. (14) Mar. 8.
The Design of and Methods and Cost of Construction by Force Account and Piece Work, a Sewerage System for a Small Town.* J. E. Tupper. (86) Mar. 12.
Difficult Sewer Construction at West Liberty, Iowa.* Charles P. Chase. (Paper read before the Iowa Eng. Soc.) (14) Mar. 15.

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Effect of Hypochlorite Treatment at Evanston. W. Lee Lewis. (14) Mar. 15.
Concrete Sewer-Pipe Tests in Kansas City.* E. S. Wallace. (13) Mar. 20.
The Garrison Creek Storm Overflow Sewer in the City of Toronto.* Ray R. Knight. (96) Mar. 20.
Temporary Hypochlorite Plant for Treating the Sewage of Norristown, Pa.* S. Cameron Corson. (13) Mar. 20.
Plumbing in a Cincinnati Bath House.* (101) Mar. 21.
Street Cleaning Methods in European Cities. (86) Mar. 26.
New System of Ventilation for School Rooms.* (101) Mar. 28.
Cincinnati Sewerage Investigations and Surveys. J. E. Root. (14) Mar. 29.
Les Huitres et la Fièvre Typhoïde à Paris.* Paul Vincey. (92) Feb.
Herstellung von Dränröhren.* C. Schimm. (80) Dec. 10.
Die Fortschritte auf dem Gebiete des Volksbadewesens in den Vereinigten Staaten. Wilh. Paul Gerhard. (7) Feb. 15.
Heizungs-, Lüftungs- und Dampfkraftanlagen in den Vereinigten Staaten von Amerika.* de Grahl. (7) Feb. 22.
Gesundheitspflege beim Bau des Panamakanals. (7) Mar. 1.
Niederdruckdampfheizung ohne Vertiefung der Kesselanlage mit selbsttätiger Rückspeisung.* Richard Wagner. (7) Mar. 1.
Die Berechnung des Wärmebedarfes zur Beheizung von Wohnräumen. C. A. Gullino. (7) Mar. 1.
Die Klärung des Abwassers in Schieferplattenkörpern.* Bach. (39) Mar. 5.
Die Entlastung des Alsbachkanales.* Heinrich Stolz. (53) Serial beginning Mar. 7.

Structural.

An Economical Design for a Timber Cotton Shed.* E. S. Pennebaker. (4) Feb.
The Painting of Iron and Steel. (12) Serial beginning Feb. 28.
Fire Protection for the Foundry. (108) Mar.
Value of Washed Sand and Gravel in Concrete.* (67) Mar.
Proportioning of Foundations for Columns and Walls. Ernest McCullough. (13) Mar. 6.
Factory Building Equipment Details.* Harry C. Spillman. (20) Mar. 6.
Hardpan and Other Soil Tests.* J. Norman Jensen. (13) Mar. 6.
Buildings for Engineering Works. H. N. Allott, M. Inst. C. E. (Paper read before the Manchester Assoc. of Engrs.) (47) Mar. 7.
The Tallest Office Building in the World, Erection of the Woolworth Building, New York.* (46) Mar. 8.
New Specifications for Structural Steel Work (American Bridge Co.).* (14) Mar. 8.
Industrial Plant with All Structures Built of Reinforced Concrete, Buildings Include Large Machine Shop with Sawtooth Construction.* W. J. Knight. (14) Mar. 8.
Grandstand for the Brooklyn Baseball Club.* (14) Mar. 8.
Building Tapering Concrete Chimneys.* D. D. Gill. (62) Mar. 10.
Methods of Molding Reinforced Concrete Shells for Pier and Wharf Foundations at the Balboa Terminals, Panama. (86) Mar. 12.
Miscellaneous Labor Costs of Excavation and Construction for a Manufacturing Plant. (86) Mar. 12.
Structural Details and Cost of Constructing a Reinforced Concrete Sand Bin.* G. A. Flink. (86) Mar. 12.
Test and Inspection of Bulb Pointed Concrete Piles for Building Foundation at North Billerica, Mass.* Hunley Abbott. (86) Mar. 12.
Action of Acids, Oils and Fats on Concrete. (13) Mar. 13.
Windbracing without Diagonals for Steel-Frame Office-Buildings.* R. Fleming. (13) Mar. 13.
The Use of White Lead in Painting. Noel Heaton. (29) Mar. 14.
The Corrosion of Aluminum. G. H. Bailey. (Paper read before the Inst. of Metals.) (11) Mar. 14.
Methods of Fire Prevention, Excellent Work Done by the British Authorities. (19) Mar. 15.
The Cost of a Brick House *vs.* One of Stucco.* Arthur W. Joslyn. (Paper read before the Building Brick Assoc. of America.) (76) Mar. 15.
Steelwork of the Broadway Arsenal, Buffalo.* (14) Mar. 15.
Reinforced Concrete Coal Trestle at Gilbertville, Massachusetts.* (14) Mar. 15.
Constancy of Volume Accelerated Tests of Portland Cement. Max Gary. (Paper read before the Inter. Assoc. for Testing Materials.) (96) Mar. 20.
Steel Forms for Concrete Construction. William Mayo Venable. (From *The Cement World.*) (96) Mar. 20.
A Non-Corrosive Iron Alloy. (13) Mar. 20.
Tests on Cast-in-Place Concrete Piles.* Francis L. Pruyn. (13) Mar. 20.
Some Short-Cuts in Reinforced-Concrete Beam Design.* M. J. Lorente. (13) Mar. 20.

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Structural—(Continued).

Imperfect Concrete Piles.* (14) Mar. 22.
Novel Concrete Stairway at a German Exposition. (14) Mar. 22.
Tests of Two Brick Piers of Unusual Size.* James E. Howard. (Abstract of paper read before the National Brick Manufacturers' Assoc.) (14) Mar. 22.
A Comparative Test of Two Fullsized Reinforced-Concrete Flat-Slab Panels.* Henry T. Eddy. (13) Mar. 27.
Layout and Building Details of a Machine Shop.* (14) Mar. 29.
The Preservation Treatment of Wood with Water Gas Tar. Frank C. Mathers. (Paper read before the Indiana Gas Assoc.) (24) Serial beginning Mar. 31.
Villa au Parc Saint-Maur (Seine).* Remson et Robine. (35) Mar.
Le Nouveau Théatre de Belleville à Paris.* (33) Mar. 8.
Die Sicherheit von Mauern und verwandten Tragwerken gegen Erddruck, Wind- und Wasserdruck.* E. Elwitz. (81) Pt. 1.
Ueber Rahmenträger und ihre Bezeihungen zu den Fachwerkträgern.* Fr. Engesser. (81) Pt. 1.
Sprengwände in Eisenbeton.* H. W. K. Ziesemer. (80) Nov. 16.
Aschenstein-Herstellung.* (80) Dec. 14.
Prüfung von Gesteinsbohrmaschinen. (80) Dec. 19.
Die Erweiterungsbauten des Getreidespeichers in Königsberg ausgeführt von Amme. Giesecke & Konegen A.-G. in Braunschweig.* M. Buhle. (48) Jan. 11.
Knickversuche mit einer Strebe des Eingestürzten Hamburger Gasbehälters.* H. Rudeloff. (69) Feb.
Die Normaleprofile für Walzeisen zu Bauzwecken vom Standpunkt der Eisenkonstrukteure; Kritik und Vorschläge.* Franz Czech. (69) Feb.
Ueber Knickfestigkeit und einseitig gedrückte Stäbe. Müller-Breslau. (69) Feb.
Ursprungsfestigkeit und statische Festigkeit, eine Studie über Ermüdungserscheinungen.* P. Ludwig. (48) Feb. 8.
Das Rosten des Eisens, seine Ursachen und seine Verhütung durch Anstriche.* Georg Pfleiderer. (48) Feb. 8.
Ueber die Wirkung des Auftriebes unter Pfeilern.* Zimmermann. (40) Feb. 15.
Der Entwurf eines preussischen Wohnungsgesetzes. A. Reich. (39) Feb. 20.
Eisenbetonarbeiten in der Volksbade- und Schwimmanstalt in Gladbeck i. W.* Paul Müller. (51) Serial beginning Feb. 22.
Beitrag zur Berechnung steifer Rahmenkonstruktionen.* A. Franke. (78) Feb. 26.
Moderne Betonwächterhäuser.* Frank C. Perkins. (78) Feb. 26.
Bau eines Zementsilos in Golleschau.* Franz Böhm. (78) Feb. 26.
Die Abbruchkosten von Eisenbetonbauten. Ernst Schick. (78) Feb. 26.
Neue Schraubenspannplatte für Gleise, Hochbau, Kranbau, Luft- und Erdfahrzeuge, Maschinen, Geräte usw. (53) Feb. 21.
Zur Statischen Berechnung des Vollwandigen Bogenträgers mit zwei Gelenken.* Siegmund Schwätzer. (69) Mar.
Der Gegenwärtige Stand des Materialprüfmaschinenbaues.* Max Kurrein. (69) Serial beginning Mar.
Berechnung gewölbter Platten.* Huldreich Keller. (107) Serial beginning Mar. 1.
Das neue Rathaus in Döbeln.* Hugo Licht. (51) Mar. 12.
Die Tischlereihalle A. M. Luther in Reval.* O. Lilscher. (78) Mar. 14.
Die Schweiz Nationalbank in Bern.* Ed. Joos. (107) Mar. 15.

Topographical.

Azimuth by Direct Solar Observation.* Louis Ross. (13) Mar. 6.
Stereophotographic Surveying.* Otto Lemberger. (13) Mar. 27.
Topographie und Reliefkunst.* F. Becker. (107) Mar. 15.

Water Supply.

Southampton's Open-Air Swimming Bath.* J. A. Crowther. (104) Feb. 28.
The Arniberg Hydro-Electric Power Station.* (12) Feb. 28.
Reinforced Concrete Stand Pipe at Belton, Texas.* Thomas L. Fountain, Assoc. Am. Soc. C. E. (36) Mar.
Failure of Nashville Reservoir.* (60) Mar.
The Georgetown Reservoir.* J. J. Bain. (100) Mar.
A Big Irrigation Dam and Koppel Cars. (67) Mar.
The Suitability of Internal Combustion Engines for Service in Small Water Works Plants. George A. Main. (86) Mar. 5.
A Discussion of the Problems Connected with the Effort to Establish Standards for the Hygienic Purity of Public Water Supplies. Joseph W. Ellms. (Paper read before the Indiana San. and Water Supply Assoc.) (86) Mar. 5.
The Danger of Physical Cross Connection Between Public and Private Water Supply Systems, A Suggested Mechanical Safeguard.* H. E. Jordan. (Paper read before the Indiana Sanitary and Water Supply Assoc.) (86) Mar. 5; (14) Mar. 29.
The Moose Jaw Water Supply. P. Gillespie. (96) Mar. 6.
The Rational Use of Water in Irrigation. John A. Widstoe. (Abstract of paper read before the National Irrig. Congress.) (96) Mar. 6.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XXXIX. APRIL, 1913. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

STATICAL LIMITATIONS UPON THE STEEL REQUIREMENT IN REINFORCED CONCRETE FLAT SLAB FLOORS.

By John R. Nichols, Jun. Am. Soc. C. E.

To be Presented May 21st, 1913.

Although statics will not suffice to determine the stresses in a flat slab floor of reinforced concrete, it does impose certain lower limits on these stresses. It is the purpose of this paper to inquire into these limiting stresses and to establish their values for comparison with those obtained by current methods of designing floors.

The nature of the limitations imposed by statics is best shown by an illustration. If we are told that three stones weigh 6 lb., this does not establish the weight of any one stone, but it does ensure that the heaviest stone weighs at least 2 lb. Therefore, although we cannot determine the value of the stress at a given section of a flat slab by statics, we can establish a stress intensity for the steel which will certainly be attained and possibly greatly exceeded. Such a quantity would point out the existing danger of overstressing the steel, even though it could not assure us of safety.

Consider the case of an intermediate panel in a floor of the ordinary flat slab type, supported on flare-topped columns, extending indefinitely on all sides, the whole floor subject to full uniform live and dead

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

load. This is the case ordinarily presupposed in the design of a floor of this type. For simplicity we will consider all the panels square, and will discuss the equilibrium of that portion of the panel which is heavily outlined in Fig. 1, a quarter-panel, omitting the area over the column top.

The forces acting on this portion of the slab are its own weight and the live load, both uniformly distributed over its surface; the vertical shear on the curved section, A; and the couples (in vertical planes) on the sections, A, B, C, D, and E.

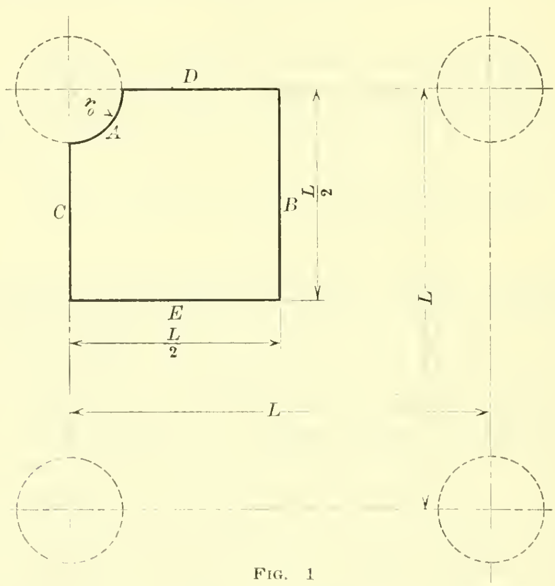

FIG. 1

The assumption that there is no shear on the sections, B, C, D, and E, is not made arbitrarily, however reasonable it may be. Not only is it a reasonable assumption, but it is the most favorable that can be made with reference to low stresses. For, if any shear existed on one of these sections, in such a way as to lessen the other stresses, this same shear acting in a reverse direction on the adjacent quarter-panel would increase correspondingly the stresses there, and we would have to give our attention to that quarter-panel which involves the greatest stresses. Obviously, the most favorable supposition is that all quarter-panels are equally effective load carriers, and this means, as just stated, absence of shear from the four straight sides.

The only outright assumption to be used as a basis for the inquiry is that the shear on the curved section, A, is uniformly distributed. The reasonableness of this assumption is obvious, and besides, a considerable variation in the shear on A involves only a slight error in the results we shall obtain, for the only use made of this assumption is in locating the resultant of the vertical shearing forces.

If the unit live and dead load is w pounds per square foot, the total load on the quarter-panel is

$$R_1 = w\left(\frac{L^2}{4} - \frac{\pi\, r_0^{\,2}}{4}\right).$$

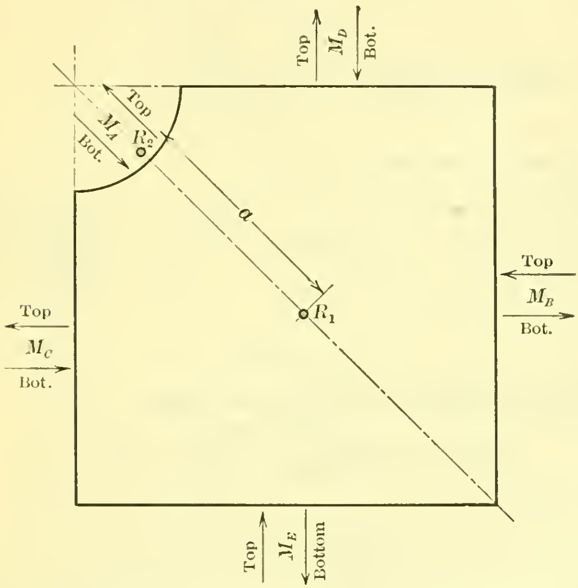

FIG. 2.

The vertical shear on A has the same value, and these two vertical forces together form the bending couple acting on the quarter-panel, which is balanced by the resisting couples on the five sides. The resultant, R_2 (Fig. 2), of the shearing forces on A, is at the center of gravity of the 90° arc at A, which is on the diagonal line from the column to the opposite corner of the quarter-panel, and is distant from the center of the column

$$\frac{2\sqrt{2}}{\pi}\, r_0.^*$$

* See any Engineers' Handbook.

The resultant of the loads passes through this same diagonal line. Its distance, z, from the center of the column is obtained as follows, taking moments about the column center:

	Area.		Distance.		Moment.
Square	$\frac{L^2}{4}$	$\times$	$\frac{L}{4}\sqrt{2}$	$=$	$\frac{L^3}{16}\sqrt{2}$
Quadrant...	$\frac{\pi r_0^2}{4}$	$\times$	$\frac{4\sqrt{2}}{3\pi} r_0$*	$=$	$\frac{r_0^3}{3}\sqrt{2}$
Difference..	$\frac{L^2}{4} - \frac{\pi r_0^2}{4}$	$\times$	z	$=$	$\left(\frac{L^3}{16} - \frac{r_0^3}{3}\right)\sqrt{2}$

$$\text{Distance to } R_1 \text{ is } z = \frac{\left(\frac{L^3}{16} - \frac{r_0^3}{3}\right)\sqrt{2}}{\frac{L^2}{4} - \frac{\pi r_0^2}{4}}.$$

The distance, a, between R_1 and R_2, is

$$a = \frac{\frac{L^3}{16} - \frac{r_0^3}{3}}{\frac{L^2}{4} - \frac{\pi r_0^2}{4}}\sqrt{2} - \frac{2 r_0}{\pi}\sqrt{2} = \frac{\frac{L^2}{16} - \frac{r_0^3}{3} - \frac{L^2 r_0}{2\pi} + \frac{r_0^3}{2}}{\frac{L^2}{4} - \frac{\pi r_0^2}{4}}\sqrt{2}.$$

We may now write

$$M = R_1\, a,$$

or, putting in the values of R and a, just found,

$$M = w\sqrt{2}\left(\frac{L^3}{16} - \frac{L^2 r_0}{2\pi} + \frac{r_0^3}{6}\right).$$

Let $r_0 = k\,L$

$$M = w\,L^3\,\frac{\sqrt{2}}{16}\left(1 - \frac{8}{\pi}k + \frac{8}{3}k^3\right)$$

where M is the couple made up of the load and the supporting shear in the vertical diagonal plane through R_1 and R_2.

The component of this couple about a horizontal axis parallel to the side, B, is

$$M_x = \frac{w\,L^3}{16}\,(1 - 2.55\,k + 2.67\,k^3)$$

or

$$M_x = K\,w\,L^3$$

where

$$K = \frac{1 - 2.55\,k + 2.67\,k^3}{16}.$$

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* See any Engineers' Handbook.

M_y, the component of M about a horizontal axis parallel to D, evidently has the same value as M_x.

Values of K for ordinary values of k are given in the diagram, Fig. 3.

Having found the attacking couple, M (or its two components M_x and M_y), there remains but to point out that the resisting moments

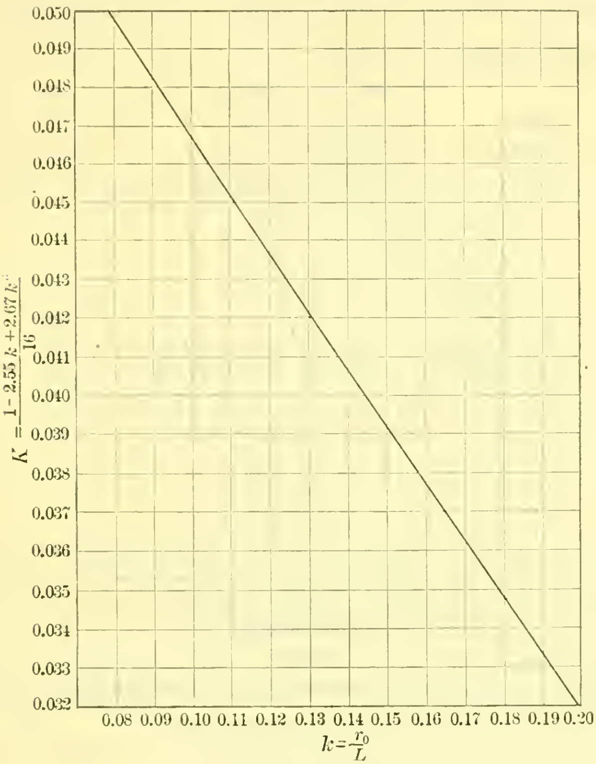


FIG. 3.

on the five sides of the quarter-panel, taken together, must be sufficient to balance M and hold this portion of the slab in equilibrium. If these resisting moments be expressed in terms of the stress in the steel, it is easy to determine whether any given design satisfies this purely statical requirement. It will be worth while to carry the inquiry a little farther and apply the results already obtained to a few simple types of reinforcement.

I.—Perhaps the simplest type consists of rods parallel to the sides of the square panels, at the top of the slab where they cross A, C, and D (Fig. 4), and at the bottom where they cross B and E. Let A_s be the total area of all the rods in both directions in a panel, d' the vertical distance from the center of the steel to the center of compression of the concrete, and f_s the unit stress in the steel. The area of cross-section of the steel in Section B is $\frac{A_s}{4}$, the total tension is $\frac{A_s f_s}{4}$, and the resisting moment on B is

$$M_B = \frac{A_s}{4} d' f_s.$$

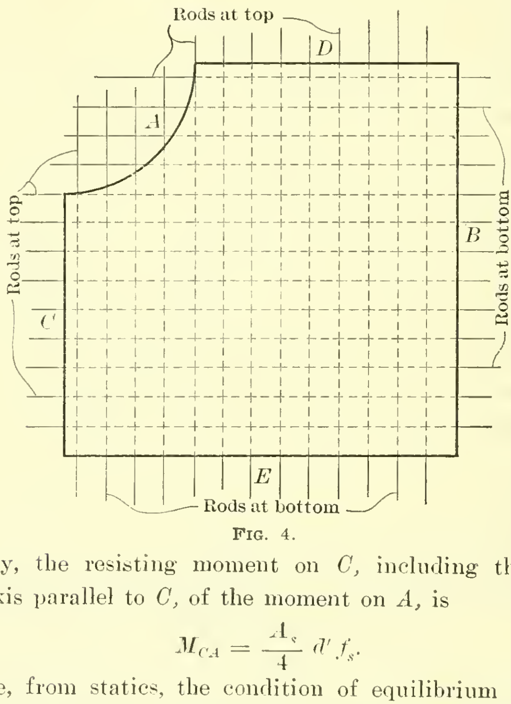


FIG. 4.

Similarly, the resisting moment on C, including the component about an axis parallel to C, of the moment on A, is

$$M_{CA} = \frac{A_s}{4} d' f_s.$$

We have, from statics, the condition of equilibrium that the sum of the moments about any axis must be zero. Applying this principle to the present case, and taking a horizontal axis parallel to C, we get

$$M_x = M_B + M_{CA}$$

$$= \frac{A_s}{2} d' f_s = K w L^3$$

or,

$$A_s d' f_s = 2 K w L^3$$

$$f_s = \frac{2 K w L^3}{d' A_s}.$$

It should be borne in mind that this is the lowest limit of the value of A_s for a given maximum f_s consistent with the fundamental principles of statics; and the maximum stress in the steel for a given A_s will not be as low as f_s, except under the conditions already assumed (and they are the most favorable possible), and under the further assumption that all the steel at all the five sections, which constitute the periphery of the quarter-panel, is uniformly stressed. If the stress intensity is less than f_s at any point, it must necessarily be greater than f_s at some other point. The stated value of f_s is certainly reached and may be greatly exceeded.

II.—If the area of the steel is doubled at the sections subject to negative bending, by lapping the rods a sufficient distance at the top of the slab where they cross the sides of the whole panel, we may write

$$M_B = \frac{A_s}{4}\, d'\, f_s, \text{ as before.}$$

but now,

$$M_{CA} = \frac{A_s}{2}\, d'\, f_s$$

$$M_x = M_B + M_{CA} = \frac{3}{4}\, A_s\, d'\, f_s$$

$$A_s\, d'\, f_s = \frac{4}{3}\, K\, w\, L^3$$

$$f_s = \frac{4\, K\, w\, L^3}{3\, d'\, A_s}.$$

III.—A type of reinforcement in common use consists of equal belts of rods in four directions, all passing over or near the tops of the columns (Fig. 5). All the rods are at the bottom of the slab midway between the columns and at the top over the columns. The pull in the rods piercing the side, B, of the quarter-panel, amounts to $\frac{f_s\, A_b}{2}$ from the half of the Belt (1) and $\frac{f_s\, A_b}{\sqrt{2}}$ from the halves of the Belts (2) and (3), where A_b is the area of cross-section of one belt. Both of these are normal to the side, B. The resisting moment on B is then

$$M_B = \left(\frac{A_b}{2} + \frac{A_b}{\sqrt{2}}\right) d'\, f_s = 1.207\, A_b\, d'\, f_s.$$

Similarly, for the steel over the column

$$M_{CA} = \left(\frac{A_b}{2} + \frac{A_b}{\sqrt{2}}\right) d'\, f_s = 1.207\, A_b\, d'\, f_s.$$

Then
$$M_x = M_B + M_{CA} = 2.41\ A_b\ d'\ f_s$$
$$A_b = \frac{K\ w\ L^3}{2.41\ d'\ f_s}.$$

The total cross-sectional area of the four belts is
$$A_s = 4\ A_b = \frac{1.66\ K\ w\ L^3}{d'\ f_s}$$
$$A_s\ d'\ f_s = 1.66\ K\ w\ \ L^3$$
$$f_s = \frac{1.66\ K\ w\ L^3}{d'\ A_s}.$$

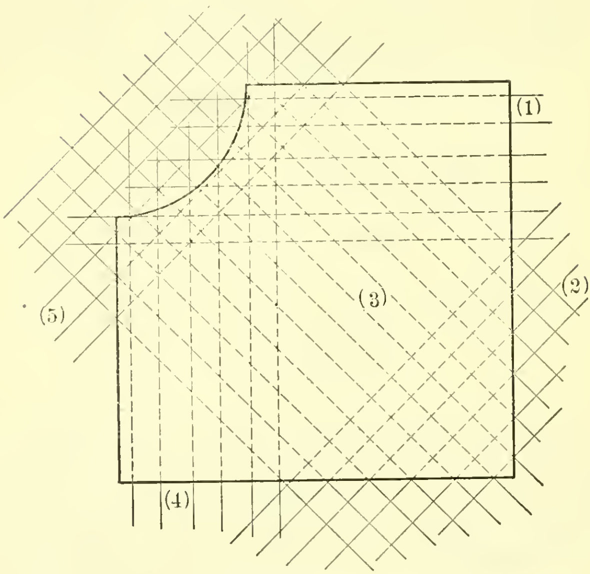


Fig. 5.

The value of A_s for this type appears to be less than that for Type I, other things being equal, but as half the rods here are 1.41 times as long, neither type shows economy over the other in weight of steel.

IV.—A fourth type is like Type III except that the diagonal rods are doubled over the column by lapping, and the rectangular or short-way belts remain in the bottom of the slab even where they pass over the columns.

Using the same notation as before, we have

$$M_B = 1.21\ A_b\ d'\ f_s \text{ as in Type III,}$$

$$M_{CA} = \frac{2\ A_b}{\sqrt{2}}\ d'\ f_s = 1.41\ A_b\ d'\ f_s$$

$$M_x = M_B + M_{CA} = 2.62\ A_b\ d'\ f_s$$

$$A_b = \frac{K\ w\ L^3}{2.62\ d'\ f_s}$$

$$A_s = \frac{1.53\ K\ w\ L^3}{d'\ f_s}$$

$$A_s\ d'\ f_s = 1.53\ K\ w\ L^3$$

$$f_s = \frac{1.53\ K\ w\ L^3}{d'\ A_s}.$$

V.—The fifth and last type of reinforcement we shall examine here is like Type III, except that all the belts are doubled by lapping a sufficient distance over the columns.

$$M_B = 1.21\ A_b\ d'\ f_s$$

$$M_{CA} = 2.41\ A_b\ d'\ f_s$$

$$M_x = 3.62\ A_b\ d'\ f_s$$

$$A_b\ d'\ f_s = \frac{K\ w\ L^3}{3.62}$$

$$A_s\ d'\ f_s = 1.105\ K\ w\ L^3$$

$$f_s = \frac{1.105\ K\ w\ L^3}{d'\ A_s}.$$

For purposes of illustration, take k at 0.10, a possible and perhaps a common value. Then $K = 0.0467$, and we may state the results of this inquiry, as applied to this case, as follows, inserting W in place of $w\ L^2$.

Type.	$A_s\ d'\ f_s$.
I.	$0.0935\ W\ L$ or $\frac{W\ L}{10.7}$
II.	$0.0623\ W\ L$ or $\frac{W\ L}{16}$
III.	$0.0775\ W\ L$ or $\frac{W\ L}{12.9}$
IV.	$0.0713\ W\ L$ or $\frac{W\ L}{14}$
V.	$0.0517\ W\ L$ or $\frac{W\ L}{19.3}$.

If $A_s\, d'\, f_s = \dfrac{W\ L}{X}$, the values of X are given for the five types in the diagram, Fig. 6.

There remains but to point out the bearing of all this on current practice in the design of flat slabs. It cannot be recalled too often that the results obtained in this paper are based solely on statics and

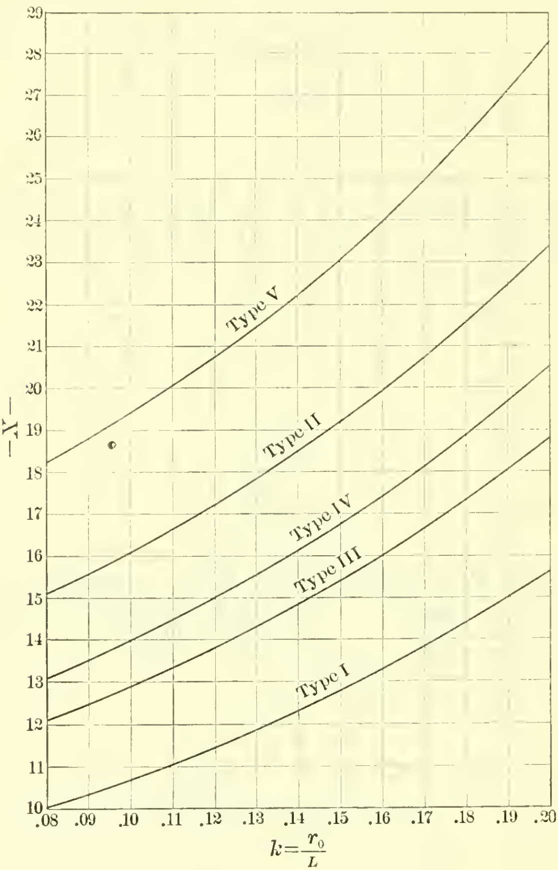


Fig. 6.

the single assumption that the shear around the periphery of the column-top is uniform. The stress obtained is not the actual maximum stress in the steel, but is the lowest value which this stress can possibly have under the loads and conditions assumed. The actual stresses

are determined by the elastic properties of the materials, and their computation. to say the least, is intricate. With whatever industry the powers of higher mathematics and the mysteries of Grashof's formulas and Poisson's ratio may be invoked, they cannot justify a result for the maximum stress in the steel smaller than the limiting value determined in this paper.

It will be said, however, that many flat slab floors are now standing under loads which, according to the results of this inquiry, imply a very high stress in the steel; and other flat slabs have been tested with loads which, if these results are correct, would seem to bring on the steel a stress in excess of its ultimate strength. It will usually suffice to discuss such evidence when offered, but one fact may be pointed out in advance, namely, that such test loads have been generally applied to single interior panels, and in this case the lateral strength and rigidity of the adjoining panels afford admirable abutments for a concrete arch or dome.

In the case of buildings in actual use, of course, it seldom, if ever, happens that a whole floor is loaded with anything approaching its full live load at one time. Here and there a panel might be fully loaded, but, through arch or dome action, a lightly loaded panel helps a heavily loaded panel as it could not do if all were fully loaded. Then again, designers have happily been in the habit of providing a factor of safety which possibly accounts for the integrity of many existing buildings. If these mitigating circumstances are to be made use of in the interest of economy, it should be done deliberately and with eyes open, not under cover of an incorrect method of design. Specifications might very well call for one live load for a single panel and another lighter load for the loading of an entire floor, thus recognizing and permitting reliance on dome action.

The method of analysis used in this paper is applicable, of course, to cases involving shapes of column top other than circular in plan, and to rectangular as well as square panels. Designs involving a deeper slab near the column than at mid-span can also be investigated by this method; but space will not be taken here to go into that. The paper will have served the writer's purpose if it makes clear the nature of the limitations set by statics upon the load-carrying capacity of flat slab floors with reference to the stress in the steel, and sufficiently

illustrates the application of the principles involved to facilitate their extension to other cases that may arise.

In closing, the writer asks those who criticize this paper not to confine themselves to citing evidence appearing to conflict with his findings, but also to point out any errors they may find in his premises or reasoning.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

RECENT IMPROVEMENTS IN LEVELING INSTRUMENTS.

BY DUNBAR D. SCOTT, M. AM. SOC. C. E.

TO BE PRESENTED MAY 21ST, 1913.

While the geodesist and the precise mechanic have been striving by independent or collateral effort to raise the art of leveling to the dignity of an exact science, it is quite evident that such investigations have induced too great a complexity in instrumental construction.

In any leveling operation the tangential axis of the bubble should be regarded as the datum plane, and the line of sight as the base line on which the survey is constructed. If the telescope is properly collimated, the line of sight, or rather the plane of vision indicated by the central horizontal wire, will occupy the equator of the field of view, and, for any ordinary distance, will describe a true horizon when regulated by a properly adjusted spirit-level of sufficient sensibility.

The fundamental requirements in any leveling instrument are simple in the essentials, but they have been made intricate by the intrusion of mechanical details between the elements on which depend the accuracy and rapidity of the operation.

The primary object is to preserve parallelism between the bubble-axis and the line of sight; and to provide an easy method of making a telescopic observation simultaneously with an ordinary observation, necessary to adjust the bubble accurately to the center of its run.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

In the Wye-level* we are required to test this most vital adjustment through the Y-supports and the collars, which, at various times, have been made of red metal, bell metal, steel, or invar, and supported on ivory or agate pivots, depending on the degree of precision sought in the work. The collars are, in reality, at the basis of adjustments, and the idea of supporting them on agate pivots, or anything else that would tend to wear them out, is only one of the fallacies experienced in the attempt to eliminate the instrumental inequalities which affect the final result.

In the Wye-level, we are also dependent, for a successful performance, on the perfect verticality of the vertical axis, which should be considered preferably only as a means to an end; for, if by some more facile method we can adjust the bubble and the sight-line to parallelism, we are certain of projecting a truly horizontal line when the bubble occupies the center of its run, irrespective of the conditions existing in the leveling base.

The practice of interrupting the connection between the telescope and the vertical axis, with a set of horizontal elements pivoted at one end and connected at the other with a "vertical adjustment" through a micrometer or gradienter screw, was first utilized by George F. Brander, of Augsburg, in 1769. This constitutes the first great and only fundamental improvement that has ever been applied to the mechanical construction of the Wye-level. It was adopted by Professor Stampfer, of Vienna, then by Kern and other European manufacturers, until to-day, it is practically the only type of Wye-level used on the Continent.

The pivots were not moved to the center of the base-bars until Ertel, of Munich, did so in 1842;† and, as we learn from Englebreit's "Instrumente der Geodasie" (Nurnberg, 1852), they were soon after elevated in the cradle-wye to the center line of the telescope where no amount of correction would have any effect upon the height of instrument.‡

The Stampfer, or the Kern model, which does not differ in essential details of construction, was first adopted in the United States

* Invented by Jonathan Sissons, of London, in 1740. See article by Daniel Eckstrom in the *Journal*, Royal Swedish Acad. of Sci., Vol. V, 1743, p. 144.

† See article by Cotta in *Dingler's Polytechnisches Journal*, Vol. 84, 1842.

‡ See also "Die Geomatrischen Instrumente," Dr. G. C. Hunäus, Hanover. 1864, p. 429.

Lake Survey, in 1876, and was used to the exclusion of other types in the U. S. Coast and Geodetic Survey until 1900.

Professor Stampfer, who applied the gradienter screw of this construction to the portable transit instrument in the early Seventies of the last century, began experiments, in the early stages of his career, by applying to all leveling operations the principle of reversion in the widest sense, not only to the instrument itself, but to the operations performed with the instrument.

In principle, the primitive diopter of ancient origin was well adapted to the theory, as will appear from inspection of Fig. 1.

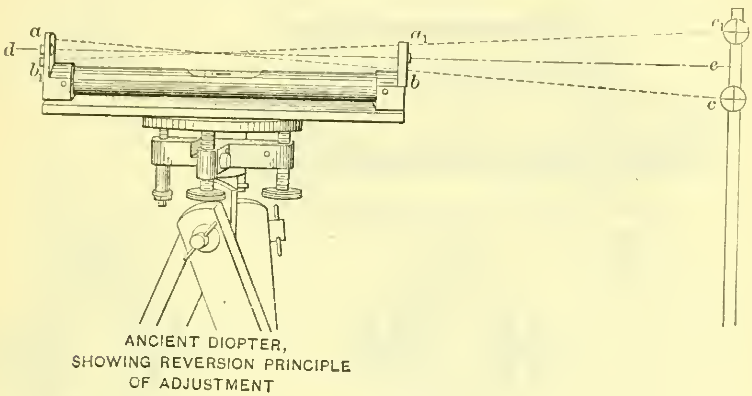


Fig. 1.

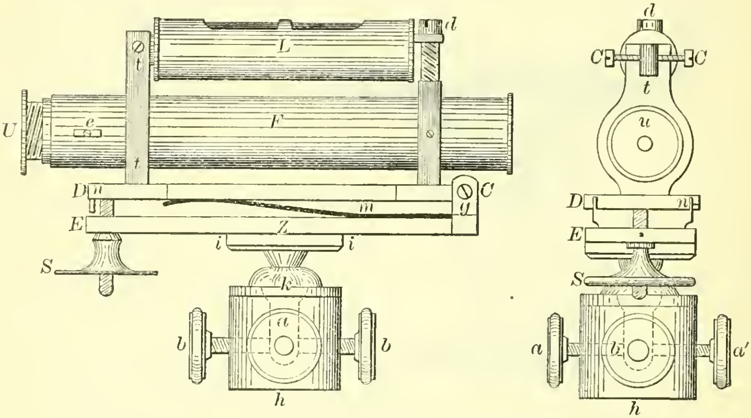
It is difficult to say where instruments of this character had their origin, but we need not look back farther than 1660 when Thevenot invented the bubble-vial. They were supplied only with peep-holes and crossed horse-hair sights, but their construction reduced the process of adjustments to the simplest possible terms.

In this case we are relieved of any necessity for perfect verticality in the vertical axis. If the first sight, which may be assumed as having been taken through *ab,* intersected the rod at *c,* on reversing the instrument, and re-centering the bubble, if necessary, we should have the second observation, as through *b′ a′*, intersecting the rod at *c,* under precisely the same conditions which controlled the first observation. Having established by this artificial means a true horizon for the height of instrument at *e*, it is not difficult to arrange the sights, *a* and *b*, into the alignment, *de,* which must be parallel with the axis of the bubble. This simple operation constitutes a most accurate and

effective means of adjustment and one which is free from the influence of any mechanical defects in the instrument itself.

In a telescopic sight, however, with a carefully collimated and unalterable sight line, we are confronted with a slightly different problem, in which the telescope must first be brought into coincidence with the mean of two such observations and the bubble finally readjusted to fit this condition.

Professor Stampfer's attempt to substitute an optical system which could be utilized for this peculiar purpose resulted in the model illustrated in Fig. 2.



STAMPFER'S DIOPTER
SIDE AND FRONT ELEVATION
FIG. 2.

The optical sight consisted of two plano-convex lenses of about 5 mm. aperture and about 32 mm. focal length, placed at opposite ends of the tube, at twice their focal length apart. Cross-wires in the common focal plane were located midway between them.

The aperture was purposely reduced to that of the pupil of the eye, for, in such a combination of lenses, the focus must be practically universal, with an emergent beam consisting of nearly parallel rays. Nothing would be gained, therefore, by producing a larger image than could enter the eye.

Each lens in turn could be used either as an eye-piece or an objective, and focusing was unimportant or rarely necessary. The focal length of the "objective" and the "ocular" being equal, the instrument possessed no telescopic power and did not elevate the problem beyond

the most commonplace consideration, except that it was possible to secure a simultaneous view of the cross-hairs and a distant target, without straining the accommodative function of the eye.

Fig. 2 is reproduced from Bauernfeind,* who says, as one would translate:

"The method utilized the most rational means of instrumental adjustment and of balancing observations, and was really capable of very accurate work. The lengths of sights were strictly limited to the range of human vision, but experience has demonstrated that its error fell within 1 : 20 000."

In the first edition of Gillespie's "Surveying" (1855), mention is made of an attempt to construct such an instrument in America, but the proposition was abandoned because there seemed to be no means of supplying telescopic power to an instrument which required optical components of symmetrical formation at both ends of the tube.

The idea was temporarily forgotten, but after discussing the matter in correspondence with W. G. Raymond, M. Am. Soc. C. E., and Mr. G. N. Saegmueller, in 1904, the writer utilized it as a theme for considerable reflection and study. Whatever may be said for the achievements of the "practical man" or the precise mechanic in the perfection of engineering instruments, this attempt to depart from the endless arguments attendant on the theory of compensating errors in all present instrumental types, was clearly a problem for the scientific optician.

An appeal was made to the best talent in both America and Germany,† and, after nearly three years of investigation in both the laboratory and field, telescopic optics of any desired power have been applied at last to the principle of reciprocal vision in the palindromic telescope herein described, which might be otherwise known as an optical Janus.

The new "Compensation Level" is shown in both elevations in Figs. 3 and 4, and in transition in Fig. 5. The writer is responsible for the details in its mechanical construction, which is, as it should be, only a means to an end—a vehicle of manipulation which does not in any appreciable manner enter into the character of the result.

* "Elemente der Vermessungskunde," by Dr. C. M. Bauernfeind, München, 1876, Vol. 1, p. 390, etc.

† *Zeitschrift für Instrumentenkunde*, Berlin, Nov., 1909, Heft 11; *ibid.*, Vol. 12, Nov., 1892, p. 377.

This instrument has a system of optics which possesses a fascination for students of both pure and applied science, comprising two achromatic objectives of equal aperture and focal length, so immovably disposed at opposite ends of the tube that their principal focii overlap, but fall slightly short of the space between them. The lens system is a perfectly symmetrical one. What is true of one direction is true of the other; the argument, therefore, will be conducted in the singular.

In all telescopes heretofore applied to engineering instruments, focusing has been conducted either by movement in the objective draw-tube, or by the ocular tube in which the diaphragm and eye-piece are mounted; and while at least the first method was attempted in the experimental stages of this problem, it has been thought desirable, above every other consideration, to fix permanently the relationship between each objective and its corresponding cross-line sight.

If the plane of the cross-wires is placed at the farthermost conjugate focal plane of the objective, we are required to concentrate a pencil of light, the origin of which may be at any position in the field, by intercepting the rays somewhere between the objective and the cross-lines, with a double concave or negative lens ground to curvatures of long radius.

This, therefore, is a lens of low sensibility, the function of which is to divert slightly the convergent rays to suit every circumstance, depending only on the position in the tube at which these rays are intercepted. An inspection of Figs. 6 and 7 will give an idea of the position of the intermediate lens, both for infinity and proximity.

In the first case (Fig. 6), rays entering the objective in nearly parallel lines, from the most distant points in the field, would naturally form an image at, or near, the principal focus of the objective at F, as indicated by the longer dotted lines; but, in passing through the negative lens, the rays are slightly bent toward the prism base, and do not come to a focus until they reach the plane of the cross-wires, as indicated at F_1.

If the incident beam enters the objective in divergent rays from a near-by object, as in Fig. 7, it would naturally converge on a conjugate focal plane, at, or very near, the fixed position of the cross-lines. In this case the negative lens is moved nearer to the diaphragm, where it will exert the least refractive influence.

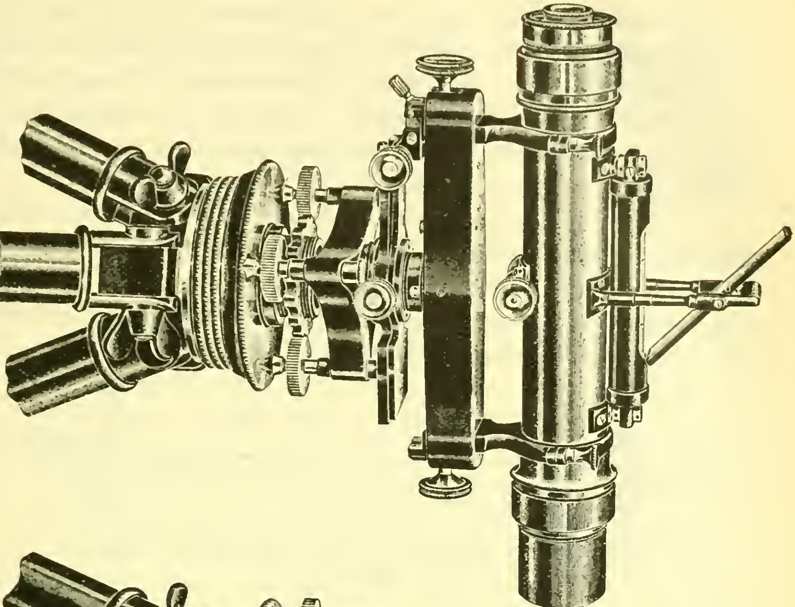


Fig. 3.—Compensation Level. Side View.

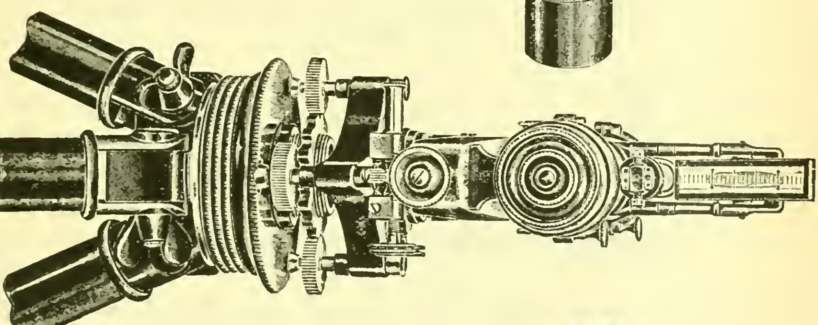


Fig. 4.—Compensation Level. End View.

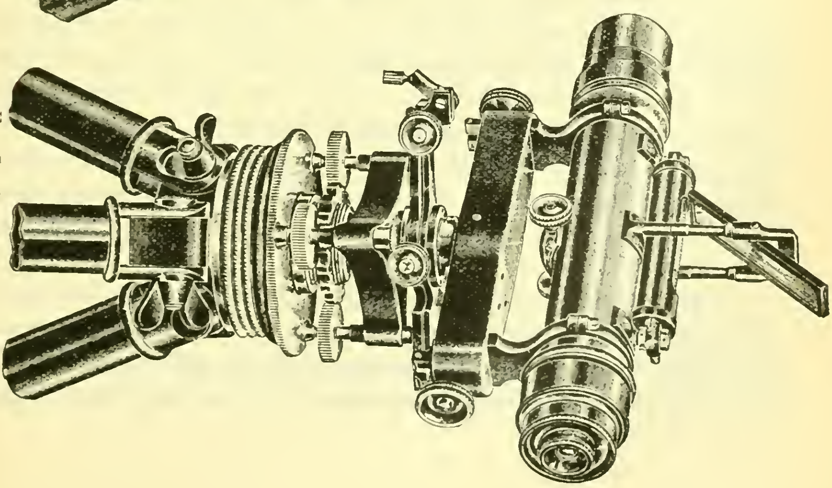
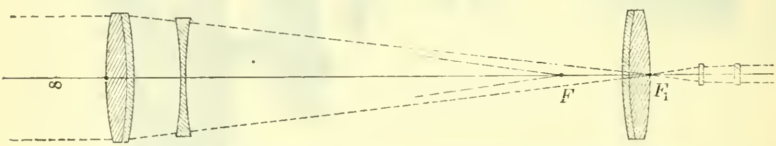


Fig. 5.—Method of Transition.

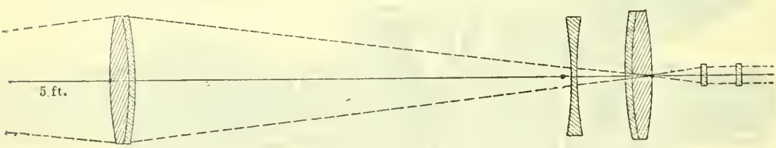
This arrangement of lenses does not preclude the use of the stadia interval, and provides a most efficient and satisfactory means of focusing, which may now be adopted in tachymeters or theodolites for mines and other such purposes, where an impervious telescope and a greater security in the collimation adjustment are of prime consideration.

As applied to leveling instruments, the intermediate lens would suggest, to a highly theoretical mind, the possibility of not only subduing the illumination, but of modifying the collimation adjustment which constitutes the one and only imperative requirement on which the accuracy of this instrument depends.



POSITION OF NEGATIVE LENS FOR DISTANT OBJECTS.

FIG. 6.



POSITION OF NEGATIVE LENS FOR NEAR-BY OBJECTS.

FIG. 7.

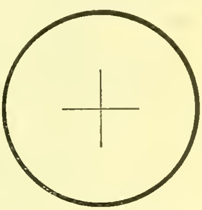
The scientific optician allows for a loss of about 8% in illumination, by reflection and absorption, in any polished lens of ordinary thickness; but this phenomenon is apparent only to a trained eye, and may be nearly corrected, without making a sacrifice to other optical qualities, by increasing the diameter of the objective 4 or 5 per cent. In this instrument, as illustrated in Figs. 3, 4, and 5, there are no more lenses than are necessary in any terrestrial telescope commonly used by the American engineer for most azimuth instruments and for many types of levels.

To favor light conditions, under these circumstances, it has been thought desirable to consider, by preference, the astronomical (inverting) ocular which is most advantageously suited for leveling instruments, whereas it might cause some annoyance in transits or theodolites.

When the target rod is used, the target itself is not changed in appearance when viewed upside down, and in self-reading rods the figures have only to be inverted to produce a normal appearance in the field of view. In general, it may be assumed that, if any level is supplied with a Ramsden, Huyghens, Kellner, Steinheil, or an orthoscopic ocular, and the rod with inverted figures, the best selection in the character of the outfit has been procured.

The optical arrangement in this instrument is such that two interchangeable astronomical oculars, producing magnifications equal to $X18$ and $X26$, may be used. This has been thought desirable, not only for the regulation of power and light, but to provide against misfortune through loss of one or the other.

Although the inverting ocular, therefore, is to be most highly recommended, it is not impossible to utilize the erecting type, as shown in connection with Fig. 10, by which a magnification of $X20$ is secured. With this eye-piece, the superlative optical qualities, with respect to illumination and size of field, cannot be expected, but it has been designed as an option for those who are confused by an inverted image. The erecting eye-piece, together with an erroneous idea that the accuracy of the sight is proportional to the length of the optical axis, are the factors which determine the unusual length of the modern Wye-level. The use of an erecting eye-piece, with the instrument under consideration as a substitute for the inverting ocular, increases the length of the telescope 40%, increases the weight of the instrument 20%, and reduces considerably the brightness of the image.



FINE CROSS-LINES ETCHED ON SURFACE OF CROWN LENS

FIG. 8.

After much experimentation with a new three-ring diaphragm (which was finally discarded), it was decided, inasmuch as the objectives were immovably and permanently fixed in the tube, that each, in turn, could be made to serve as an excellent substitute for a glass diaphragm for the other optical system.

With this in view, it became necessary to design an elaborate collimating apparatus especially for this instrument. Where the optical axis of each objective passes through the other, a microscopic cross is etched on the outer surface of the crown lens, with a fine

diamond point, somewhat as shown in Fig. 8. If the elements which enter into the collimation adjustment, therefore, are fixed, we may rationally assume a permanency in the adjustment itself, and look only to the interior focusing lens for such derangements as are possible under ordinary conditions of work.

The diaphragm opening in which the cross-lines appear is necessarily, in this case, equal to the full aperture of the objective. This condition makes it impossible to margin off the extreme outer rays which produce a secondary spectrum, but the field of view is much larger than that which is utilized by either of the eye-pieces, and a slight fringe of color in the periphery is not noticeable, unless a deliberate search is made. The working part of the field of view is free from spherical or chromatic aberration, and possesses all the optical qualities which satisfy the most fastidious demand. There is nothing, however, to prevent the construction of a diaphragm of proper proportions as a part of the eye-piece mount, and so fulfill in this particular the uttermost requirement in the case.

The position of the plane of the cross-lines has been located arbitrarily where an image will be naturally produced by rays having their origin 5 ft. beyond the objective. For the shortest sight possible to be observed through either end of this instrument, 1.5 m. has been selected.

The collimation adjustment consists of the fixation of the intersection of the cross-wires into the optical axis of the objective, as accomplished by some improvised axis of longitudinal revolution. The collimation line in this, or any ordinary, telescope may be slightly removed from this geometrical axis, depending on the amount of eccentricity which exists in the objective mount; but it will be parallel in any case.

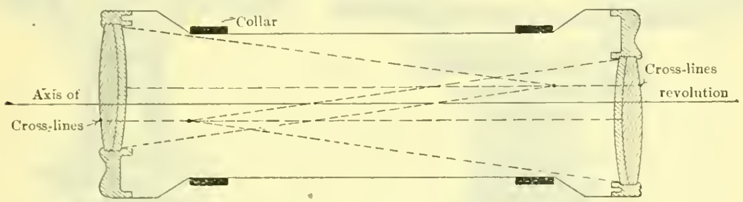
A thorough investigation into the probable causes of collimation error, in reconciling one optical system with the other, induces the conclusion that the maximum total error in reading must be equal only to the amount of eccentricity in the objectives, which is not a progressive error in any event, and, if it occurs in the horizontal plane, is a negligible quantity, however great.

With the facilities of manufacture now at the disposal of the precise mechanic, we shall perjure the premises if we allow that, in mounting, a greater displacement than 0.1 mm. may occur on either

side of the geometrical axis of the tube. Treating, however, with a most exaggerated case of this character, we shall have, after the separate adjustments for collimation, the conditions expressed in the diagram, Fig. 9, showing two possible sight-lines that must be either coincident with, or parallel to, the axis of revolution.

Eccentricity of mounting will reveal itself, after the collimation adjustment, if the telescope is revolved in the wyes. If the field of view appears to move in a small circle, there is eccentricity of mounting; but, if the collimation adjustment has been properly made, the intersection of the cross-lines will appear to move with the field.

There will be no such thing as an inclination of the optical axis in the tube by virtue of an inclination in the objective itself. No error can possibly occur through oblique mounting, except that of astigmatism, but this is incidental rather than vital, and need not be considered in this, any more than in any other, instrument.



EFFECT OF ECCENTRIC MOUNTING ON COLLIMATION.

FIG. 9.

The chances for error induced by any lateral movement in the focusing lens have to deal first with the collimation adjustment (or the position of the optical axis as fixed by the cross-lines), then with the reproduction of the rod at the plane of the cross-lines, thus fixed.

In testing the accuracy of the collimation adjustment for either optical system, some very distant point in the horizon should be selected. If the test is made on a near-by object, it will not fulfill, in the highest theoretical sense, the requirements in the case. The infallible test for errors of this character is a check of the collimation on a near-by object after perfecting the adjustment on some very distant point. This necessitates moving the interior lens between the positions shown in Figs. 6 and 7; and this test ought to qualify, irrespective of eccentricities in mounting, as previously shown by D. A. Molitor,* M. Am. Soc. C. E.

*Transactions, Am. Soc. C. E., Vol. XLV, 1901. p. 45.

An excellent collimator apparatus for use in the field may be arranged by placing the objectives of a level and a transit in juxtaposition so that a beam of light can pass directly through both optical systems. If the transit telescope is racked down to its shortest length, it will be necessary to focus the telescope of the level at, or near, infinity, in order to see the cross-wires of the transit diaphragm. This method brings an object, apparently at an infinite distance, into very convenient range, and the test may be made at night by placing a bright light immediately behind the transit eye-piece. The arrangement, as illustrated in Fig. 10, shows the level equipped with an erecting eye-piece and a special sunshade which may be used interchangeably with the inverting eye-pieces, and thus convert the telescope almost instantly from one type to the other.

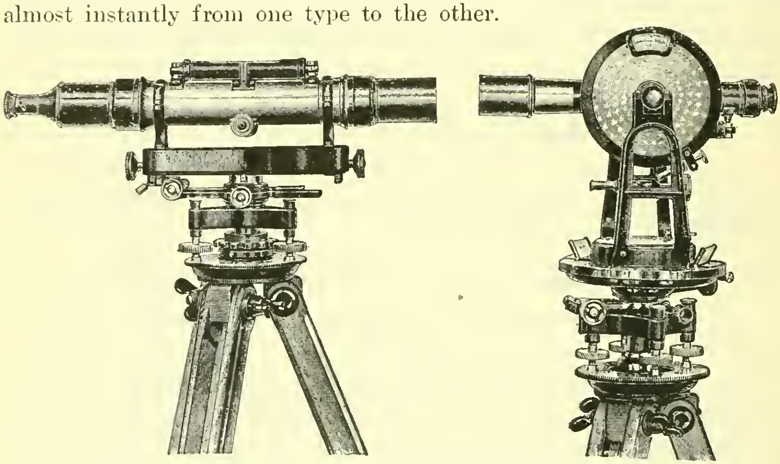


FIG. 10.—FIELD COLLIMATOR APPARATUS, SHOWING READING MIRROR REMOVED AND BEARING CLIPS LOOSENED TO FACILITATE LONGITUDINAL REVOLUTION.

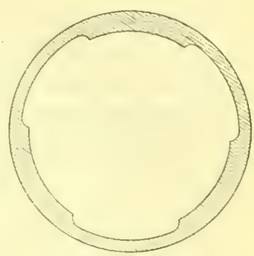
The axis of revolution is established as between the bearings in the supports. It is desirable, therefore, that the caps should be only slightly loosened and that the telescope should not be removed from its supports, or reversed for the second test, except on the vertical axis. In making the collimation test by longitudinal revolution, it is not absolutely necessary that the telescope should be horizontal; but the axis of revolution in the wyes must not be modified except as stated.

The possibility of securing permanently the collimation adjustment by fixing immovably the relationship between the elements which constitute that adjustment, commends itself at once, and, now that we

may provide against any movement for either the objective or the diaphragm, we may mount them in a tube which is sealed against the intrusion of dust or moisture.

The telescope tube has been reinforced with three longitudinal ribs, cast, drawn, and turned down in one solid piece with the outer wall. The carefully machined ribs add lateral strength, prevent flexure, preserve alignment, and form a convenient skidway on which to slide the barrel in which the focusing lens is mounted. The contact surfaces between these ribs and the lens-barrel are the only movable parts on which the accuracy of the instrument depends.

If we safely allow that the error due to eccentric mounting of the objectives is a constant not exceeding 0.2 mm. for all distances, it may be regarded as a negligible quantity, and the proposition seriously concerns us only so far as irregular motion in the focusing device may affect the reproduction of the field of view at the plane of the cross-wires.



CROSS-SECTION OF TELESCOPE TUBE

FIG. 11.

This influence has been accorded a most careful theoretical and practical investigation, with results so satisfactory that it is recommended as superior in point of accuracy to either of the other methods of focusing now in common use.

A theoretical consideration of the effect produced on the optical axis by the negative focusing lens is one involving some of the simple laws of refraction. For the purposes of this demonstration, Fig. 12 represents an exaggerated condition of affairs.

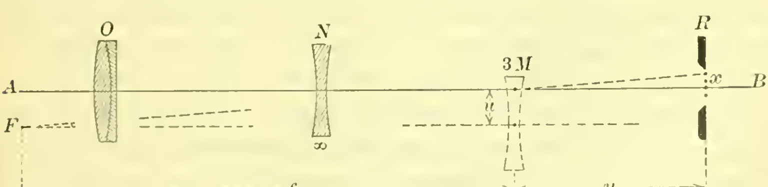


DIAGRAM SHOWING EFFECT OF LOST MOTION IN FOCUSING LENS.

FIG. 12.

Let it be assumed that an objective, O, and a reticule, R, are immovably placed with respect to each other. It is assumed that the

negative lens, being in the position, N, when focused on infinity drops out of alignment from the optical axis, AB, to the position, $3M$, through a displacement equal to u, when focused on an object, say, 10 ft. away.

In the position, $3M$, the axial ray from A will be deflected at an angle having a base equal to x, and the anterior focal point will drop accordingly to F. Let the total amount of deflection at the diaphragm be equal to x, and let the distance from F be equal to f_2. From similar triangles we have:

$$x : u :: y : f_2$$

$$x = u \frac{y}{f_2}.$$

In the very portable instrument under discussion, the focal length of the negative lens, f_2, is equal to 350 mm., and the mean value of y is equal to about 70 mm. Substituting, we have:

$$x = u \frac{70}{350} = \frac{1}{5} u.$$

In other words, for a level of this length, the displacement of the image at the diaphragm will be equal to about one-fifth of the deviation of the focusing lens from the optical axis.

The contact bearings of the inner lens-barrel can be made to move so accurately that, whether we consider the error of displacement, as applied to collimation adjustment or accuracy of pointing, we can only conclude that it must fall within the diameter of the sight line itself, and remain within the limbo of intangible quantities.

The mechanical process of focusing by this method necessitates only a movement of the interior lens between the telescope supports, where such a reapportionment of weight will not sensibly affect the center of gravity or destroy the equilibrium of the instrument.

The bubble tube is rigidly attached directly over the top of the telescope. It has adjustments for height and alignment, but the lateral adjustment is not now important, inasmuch as we have dispensed with the function of the wyes, except as a means for testing collimation. On this account we are neither concerned with the influence exerted by inequalities in the collars, due to wear or concretions of dust, nor the necessity of a striding level with which such errors may be tested.

If temperature fluctuations have any effect on the horizontality of the level vial, the elimination of such discrepancies is accomplished only by the process of end-for-end reversals, which forms a part of the prescribed routine in the performance of precise leveling with an instrument of this type.

The accuracy of results depends entirely and directly on the relation of the bubble axis to the amphidioptric sight line, and sudden changes in temperature will affect such work only as they are capable of disturbing this relationship.

H. M. Wilson, M. Am. Soc. C. E., concurs with Professor Molitor in the opinion that precise leveling does not differ materially from the ordinary kind, except in the character of the instrument; and that practically all residual errors are attributable to temperature changes.* It would seem, therefore, if we provide in the new instrument a ready means of testing the collimation, that temperature effects will compensate themselves on reversals, and that it will be no longer necessary to equalize forward or back-sights, unless it is thought desirable to do so on account of curvature and refraction.†

Although variations in temperature may affect somewhat the angular value of the graduations on the bubble vial, its capability to indicate a truly horizontal setting is not impaired if the same central segment of the graduated scale is used, as advocated by the late J. B. Johnson, M. Am. Soc. C. E.

At each side of the bubble tube are sockets for the reception of the mirror supports. The mirror frame is hinged at the center, so that the reflected image of the bubble may be viewed from either end of the telescope with equal facility. This method of observing the bubble was adopted on account of its peculiar fitness to the requirements of the case; but a stop has been provided that regulates the amount of inclination to approximately 40° each way, and also to prevent cracking the vial by reckless use.

Naturally, the longer the bubble the greater the amount of error, due to parallax in reading it by this method; but this constant, in a sensitive bubble, must necessarily be very small, and will neutralize

* *Transactions*, Am. Soc. C. E., Vol. XLV, 1901, p. 126.

† See L. S. Smith, M. Am. Soc. C. E., in *Bulletin*, Univ. of Wis., Vol. I, No. 5, 1895. W. S. Williams, Assoc. M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. XLV, 1901, p. 176. "Plane Surveying," Professor J. C. Tracy, 1898, p. 273, etc.

itself in reversals. For ordinary leveling operations it may be disregarded entirely. It would seem, from the peculiar conditions and the requirements in the case, that by the process of end-for-end reversals every real or imaginary error, traceable to the bubble or its mounting, will be perfectly compensated and automatically corrected.

Our total anxiety, therefore, in the use of the instrument may be concentrated on the parallelism between the bubble axis and the biaxial sight-line to which it must be adjusted; but this can be tested at any time, within from 3 to 5 min., in a second observation, without moving out of one's tracks and without an assistant.

For ordinary leveling purposes, the operator will naturally make this test in the morning, as required of all assistants in the Coast Survey, and, when satisfied, he will apply the eye-piece to one objective and the sun-shade to the other, for continuous and accurate work. This instrument was designed only for use in more ordinary work, but the object of this paper is to show that the construction is such that the most precise results may still be secured by two sets of double observations, on the assumption that the collimation and the bubble are both out of adjustment.

The instrument's first claim on the attention of the engineer is the wonderful facility with which the adjustment for parallelism may be tested, and the accuracy and speed with which it may be rectified. An unbiased, open-minded examination into those parts of the instrument which might be suspected of infidelity, develops the conclusion that, one by one, they may be reduced to negligible quantities.

The vertical adjustment device is an entirely new model, peculiarly adapted to the reversible qualities of this instrument. The upper element of the base-bar forms a channel encasement for the lower element, which is rigidly attached to the vertical axis.

The telescope and bubble are mounted on the upper element in non-adjustable supports. The upper element, with telescope and bubble, revolves through short arcs in the vertical plane, on a conical pivot or an eccentric worm gear at the center of the bars, some 55 mm. below the axis of the telescope.

This is an improvement over the method of placing the pivots at one end, but nothing essential would be gained by a further modification of the mechanical equipment, in order to get the pivots in the horizontal plane which is coincident with the longitudinal

axis of the telescope. The reversion principle in this level demands that we shall place the pivots at the center, in order that the height of instrument shall not be modified in the second observation. If the instrument, considered in this paper, was intended for the measurement of small vertical angles by a micrometer screw, it is agreed that the longer base secured by placing the pivots at the opposite end would contribute to the efficiency of the construction; but in the 10-in. (25-cm.), instrument herein described, the micrometer is not adaptable, and it is proposed, further, that the micrometer method of securing precise results shall be superseded by the process of reciprocal vision herein described, or by the three-wire method already adopted.

The mechanical construction of the "vertical adjustment" in this design cannot be adapted for use as a gradienter because the value of one division on the drum would change constantly in the different positions of the eccentric gear. For convenience of manipulation, we are required to communicate a slight vertical movement through a horizontal bar revolving on its own axis.

The horizontal bar terminates in milled-heads, symmetrically placed directly under each successive position of the eye-piece, so that, whether in direct or reverse position, the final delicate setting of the bubble can be accomplished without inconvenience or confusion. This method is quite unique, and is well adapted to this construction.

The upper base-bar is not always necessarily parallel with the lower, nor perpendicular to the vertical, axis. A normal position, however, may be quickly determined in reversals on the vertical axis by correcting half of the bubble displacement in the leveling base and the other half in the "vertical adjustment." In one of two such trials the bubble should remain centered during an entire revolution.

The mechanical separation of the connection between the telescope and the vertical axis distinguishes this instrument at once from the dumpy type, and places it formally in the precise class, irrespective of its numerous other claims to this distinction.

On precise levels of the Coast Survey type, a circular-box bubble has been attached to the lower element of the base-bar to secure a nearly vertical setting of the vertical axis, without regard to the relationship of the upper portion of the instrument, and independent of the more sensitive vial which is mounted on the telescope, but in the comparatively small instrument herein described a "normal adjustment"

is of such easy accomplishment that it has seemed preferable to dispense with the circular bubble, which has no given sensibility or means of adjustment.

This alternative seems the more to commend itself inasmuch as the "vertical adjustment" is constructed on an endless worm gear in which the total amount of elevation or depression cannot exceed 1.8 mm. on either side of the normal line. If either milled-head were turned continuously, the telescope would only rise and fall in uninterrupted oscillation. The operator will become quickly accustomed to the effect produced in the mirror by turning the milled-head either to the right or left, and this effect will be the same when viewed from either end of the instrument.

The tangent release mechanism is also a new idea, absolutely without precedent, so far as the writer knows. As shown in Fig. 5, the nose-piece, instead of being rigidly attached to the lower bar, is removable and adapted to engage a spring-slot at either end of the instrument, as shown. In reversing the instrument, the clamp-screw to the vertical axis need not be thrown open, as usual, but the whole clamp-and-tangent arrangement may be allowed to remain, so that it can be manipulated with the right hand in either position of the upper portion of the instrument.

When looking through either end of the telescope, the focusing screw, the vertical adjustment, the clamp screw, and the tangent screw, may be manipulated by either the right or left hand, as desired, depending on the habits of the operator, and, in fact, all other details in both the optical and mechanical equipment are symmetrical, interchangeable, and in perfect equipoise.

The type of three-screw base shown in the various illustrations is also a new model, which is attachable to the ordinary form of tripod, is applicable to the ordinary type of transit or level, and is provided with the regular type of shifting center such as the American engineer has used since 1858. For leveling instruments, the shifting center is not necessary, but in azimuth instruments, which are required to be centered over a given point, the appliance is almost indispensable. Referring to Fig. 13, the spring, which accommodates a universal movement through the center of the instrument, is no longer attached to an awkward spring-bar of the European model, but snugly contained in the central capsule of the shifting plate.

Every movement in each of the three leveling screws is compensated automatically at the center, and the tension in the leveling arms is equalized. Unequal strain in varying temperature, which has a subtle influence on the bubble, is eliminated, and the longer radial arms, with larger thumb-screws in German silver, provide a more sensitive control. This type of leveling base, while very convenient, is not indispensable to the instrument herein described, or to any other provided with a "vertical control," but, to the ordinary dumpy level, it adds greatly to ease of manipulation and efficiency of operation.

While in use, the coarsely-knurled clamp ring is to be loosened, but, for transportation, it is to be screwed down against the flange, in order to lock the instrument securely against the lower base.

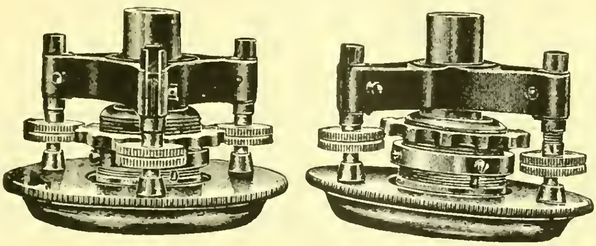


FIG. 13.—NEW THREE-SCREW BASE WITH SHIFTING CENTER.

One quickly becomes used to the three-screw system, and it is not too much to say that, if one of the screws is placed directly beneath the intersection of the two lines marking the longest axis of the bubble tubes in a transit, the bubbles may be centered at one end at the same time with remarkable speed.

In leveling up such an instrument as described, first secure the normal position of the bubble, then place the telescope parallel to any two of the leveling screws. After centering the bubble in this position, revolve the telescope 90° and center the bubble once more, using the third leveling screw alone.

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PAPERS AND DISCUSSIONS

TIDAL PHENOMENA IN THE HARBOR OF NEW YORK.

By H. de B. Parsons, M. Am. Soc. C. E.

To be Presented May 7th, 1913.

The Harbor.

For the purpose of this paper, the harbor of New York is taken to include the Lower Bay, the Upper Bay, the Hudson River to Mount St. Vincent, the East River to Throgs Neck, the Harlem River, Newark Bay, the Kill van Kull, the Arthur Kill, and Jamaica Bay. Each of these divisions can be considered a distinct branch, although the tidal phenomena, taken as a connected whole, are affected by the characteristics of each.

A map of the harbor, Fig. 1, shows the positions of these different parts and their relations to each other.

It is difficult to describe the tidal actions clearly and concisely because the movements of the water in all parts of the harbor are not synchronous. This is due to the following causes:

1.—The harbor is so extended that there is a considerable difference in time between the periods of high water at various points;

2.—The variation in the range of tides;

3.—Some of the currents being produced by progressive tidal wave motion, some by hydraulic conditions (that is, difference in water levels), and some by the interference of two distinct tidal waves;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

4.—The variable flow of fresh-water into the harbor from the rivers (that is, seasonal variation);

5.—The under-run, caused by the inertia of the flowing stream, the difference in specific gravity between sea-water and land-water, and possibly by the configuration of the harbor bottom.

Average conditions exist only occasionally. Variations in tidal range at any station are caused by the relative positions of the moon and sun, and by other factors. The winds also have a great temporary influence on the rise and fall of the tides. Outside of temporary influences, such as wind, the regular changes are periodic; therefore, the difficulty of describing the phenomena will be apparent, because the conditions which exist on one day will not exist on the days preceding or following. As the general phenomena will depend closely on average conditions, the tidal phenomena described herein will be based on mean conditions of tidal range, velocities, and volumes of tidal flow.

Where velocities have been observed on a given date they have been corrected to what they would have been had the mean conditions existed on that date. The method for the correction of velocities is the multiplication of the observed velocity by the ratio of the mean tidal range to the tidal range on the day in question, in accordance with the conditions given for the different parts of the harbor, as stated under the heading "Effect of Tidal Range on Velocity," page 670.

The water gradually shallows toward the shores. If the water surface was lowered, the boundary of the land would be changed in form. These changes are interesting and instructive, in studying the tidal phenomena of the harbor, and are shown in Figs. 2, 3, and 4, in which the water surface is drawn at mean low water, and at 6 ft. and 12 ft. below this level, respectively.

These figures show that, of the aggregate of water surfaces at mean low water, only about 58% has a depth of more than 12 ft. If the water level was lowered to 24 ft., the only water surfaces left would be the Hudson and East Rivers, which uniting just below the Battery, would flow through the main channel to the Atlantic Ocean. That is, only the important channels would remain. Taking the harbor divisions as defined in Appendix A, the water surfaces at different planes would be approximately as given in Table 1.

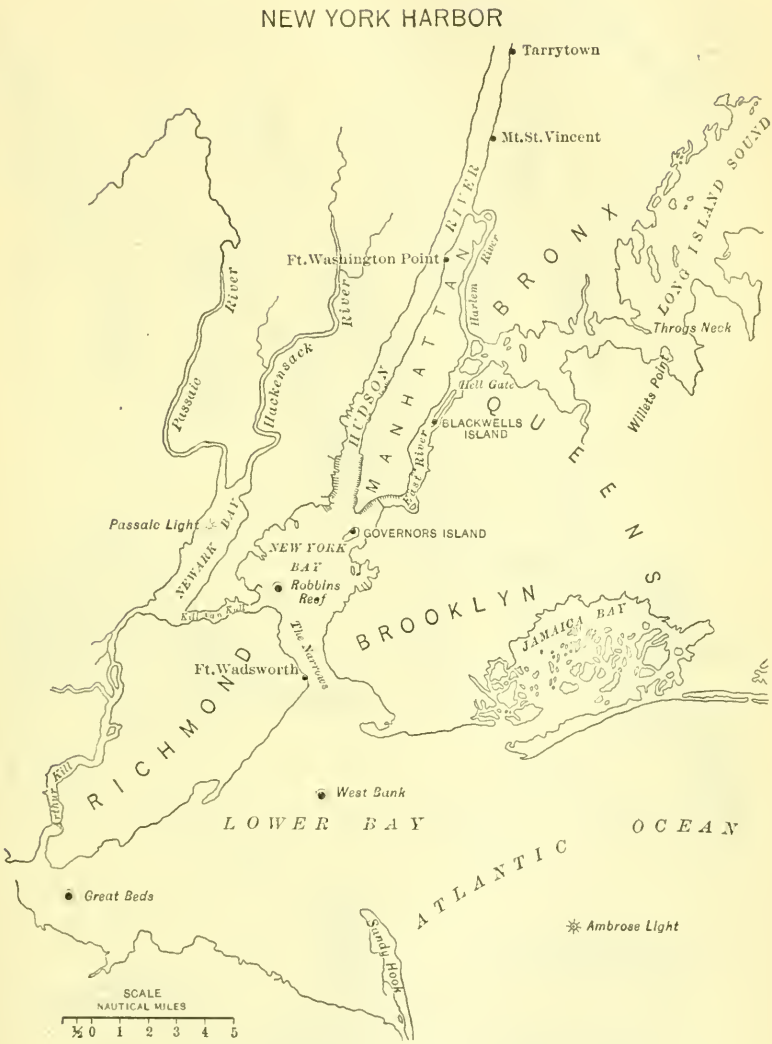
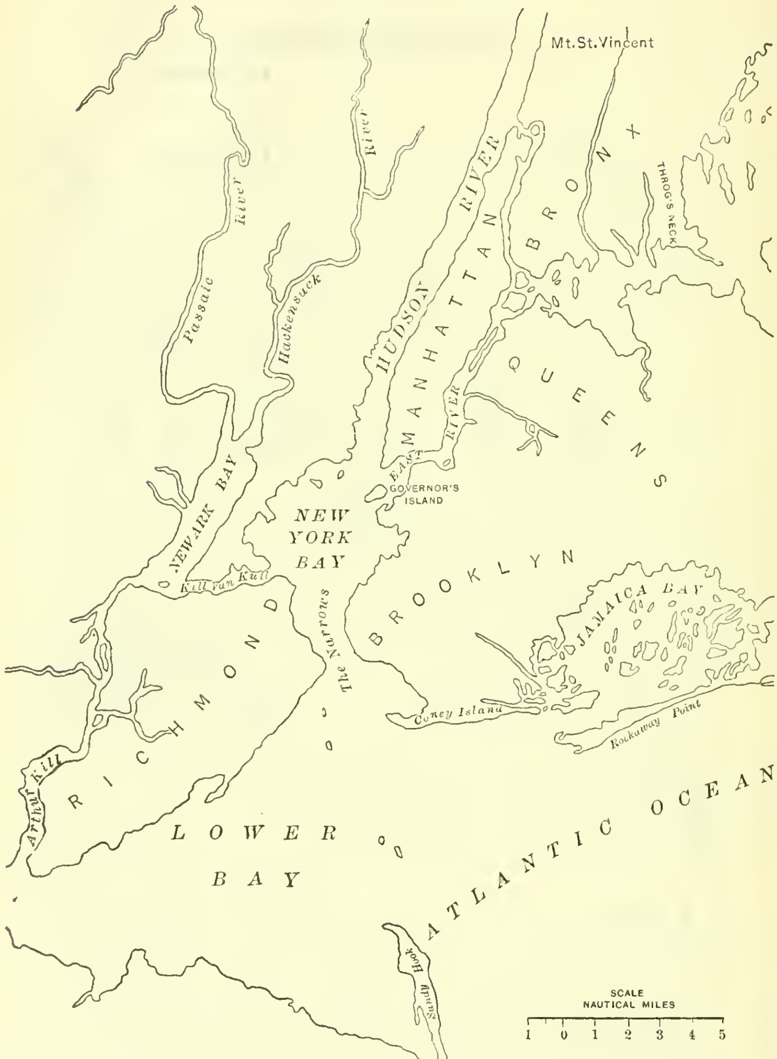
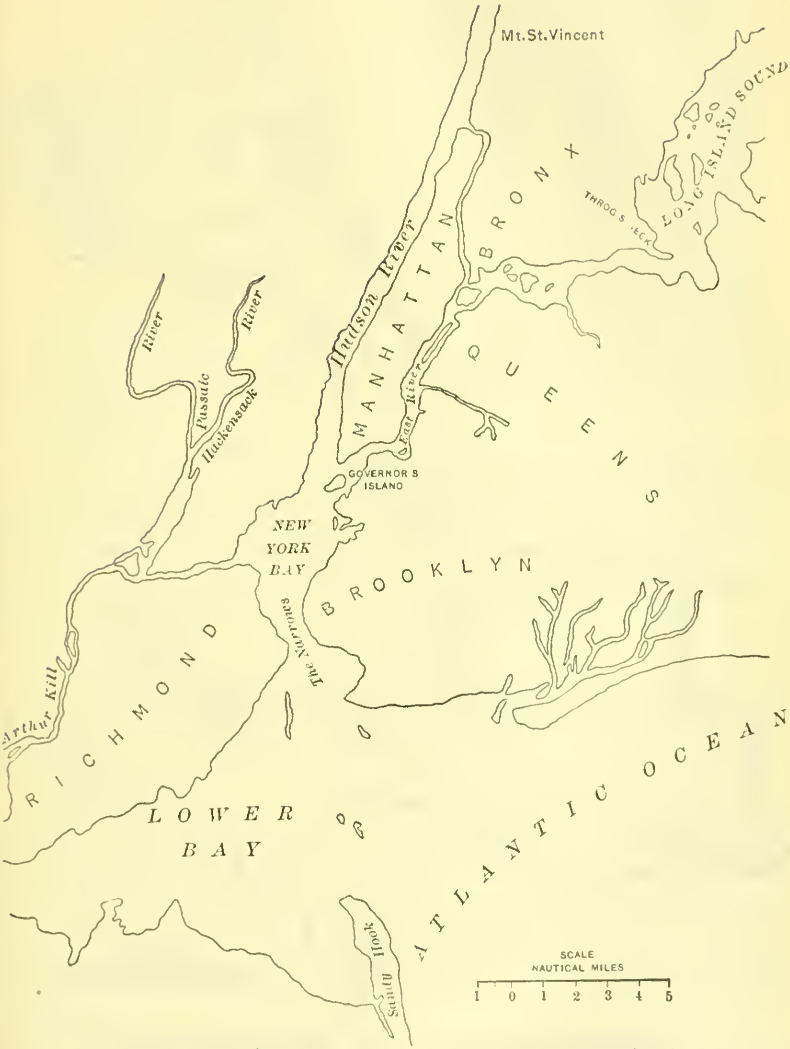

NEW YORK HARBOR
Tarrytown
Mt.St.Vincent
Ft.Washington Point
HUDSON RIVER
MANHATTAN
Harlem River
BRONX
LONG ISLAND SOUND
Throgs Neck
Willets Point
Hell Gate
BLACKWELLS ISLAND
East River
QUEENS
Passaic River
Hackensack River
Passaic Light
NEWARK BAY
GOVERNORS ISLAND
NEW YORK BAY
Robbins Reef
Kill van Kull
The Narrows
Ft.Wadsworth
BROOKLYN
JAMAICA BAY
RICHMOND
Arthur Kill
West Bank
LOWER BAY
OCEAN
ATLANTIC
Great Beds
Ambrose Light
Sandy Hook
SCALE
NAUTICAL MILES
½ 0 1 2 3 4 5

FIG. 1.



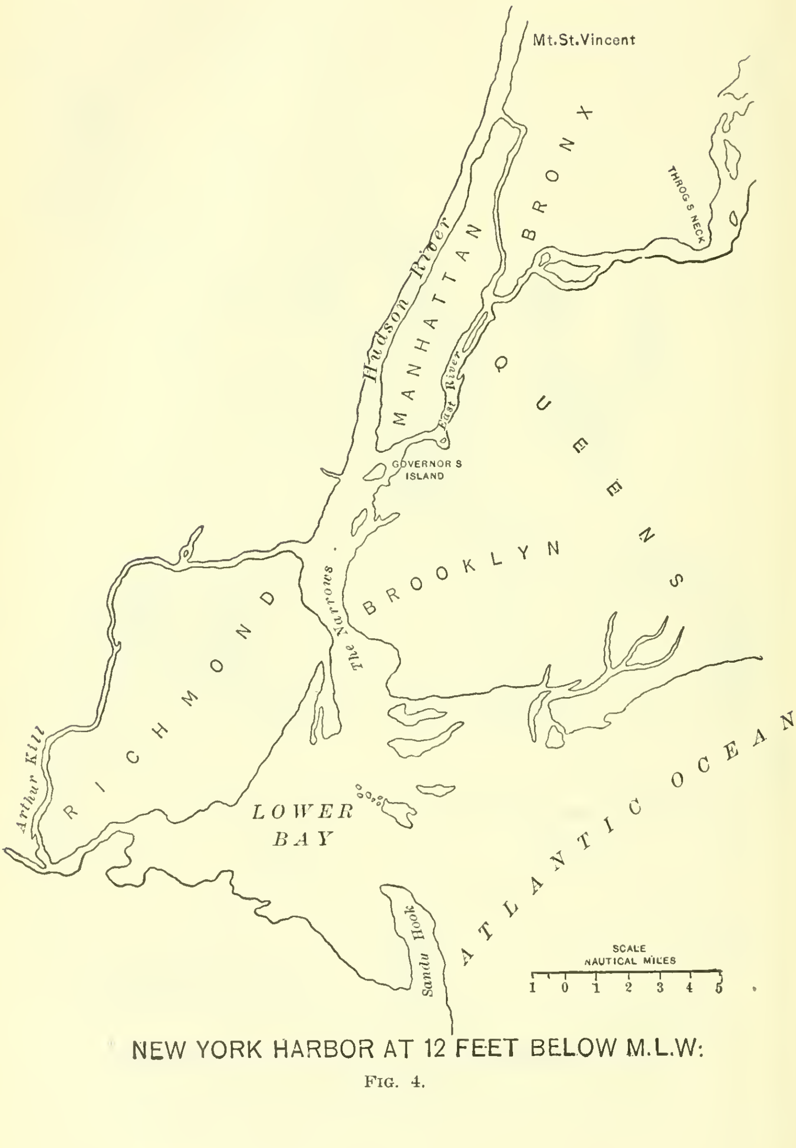
NEW YORK HARBOR AT MEAN LOW WATER

FIG. 2.



NEW YORK HARBOR AT 6 FEET BELOW M.L.W.

FIG. 3.



NEW YORK HARBOR AT 12 FEET BELOW M.L.W:

FIG. 4.

TABLE 1.—AREAS OF WATER SURFACES, IN SQUARE MILES.

Part of harbor.	At M. L. W.	At Depths Below M. L. W.	
		6 ft.	12 ft.
Lower Bay	124.30	106.20	83.60
Upper Bay	20.74	14.88	12.36
Hudson River, Battery to Mt. St. Vincent	14.49	13.73	12.88
East River, Battery to Throgs Neck	15.20	10.30	8.54
Harlem River	0.59	0.47	0.38
Newark Bay	8.35	3.28	0.64
Kill van Kull	1.12	0.97	0.88
Arthur Kill	4.63	3.34	2.31
Jamaica Bay, excluding islands	21.61	4.40	2.20
	211.03	157.57	123.79
	100%	74.6%	58.6%

The phenomena of "under-run" and "inertia" are well confined within areas having a depth of water greater than 12 ft. below mean low-water level.

Profiles of the harbor bottom are shown in Figs. 5 and 6. The former is a profile through the Lower Bay, Upper Bay, East River, and Hudson River, along the line of the channel; the latter is a profile of average depths of sections across the stream along the same line. Between the Atlantic and the Sound entrances of the harbor, the depths of the sections in general tend to vary inversely as their widths.

Tidal Currents.

The normal currents in the harbor are produced by progressive or periodic waves, by hydraulic conditions, or by a combination of these two causes.

The progressive wave is the natural motion, due mainly to the attractions of the moon and sun. When the wave is unobstructed, the maximum flood and ebb velocities occur at about the times of high and low water, respectively. The profile of a true tidal wave is a sinusoid. The velocities, as well as the heights (stages) of the water, are proportional to the ordinates of a sinusoid the abscissas of which have time values.

When hydraulic conditions obtain, a current flows, on account of the difference in head which exists temporarily between the bodies of water connected. In this case the maximum velocities occur at some time between local high and low waters, depending on the time of greatest hydraulic slope and the characteristics of the channel.

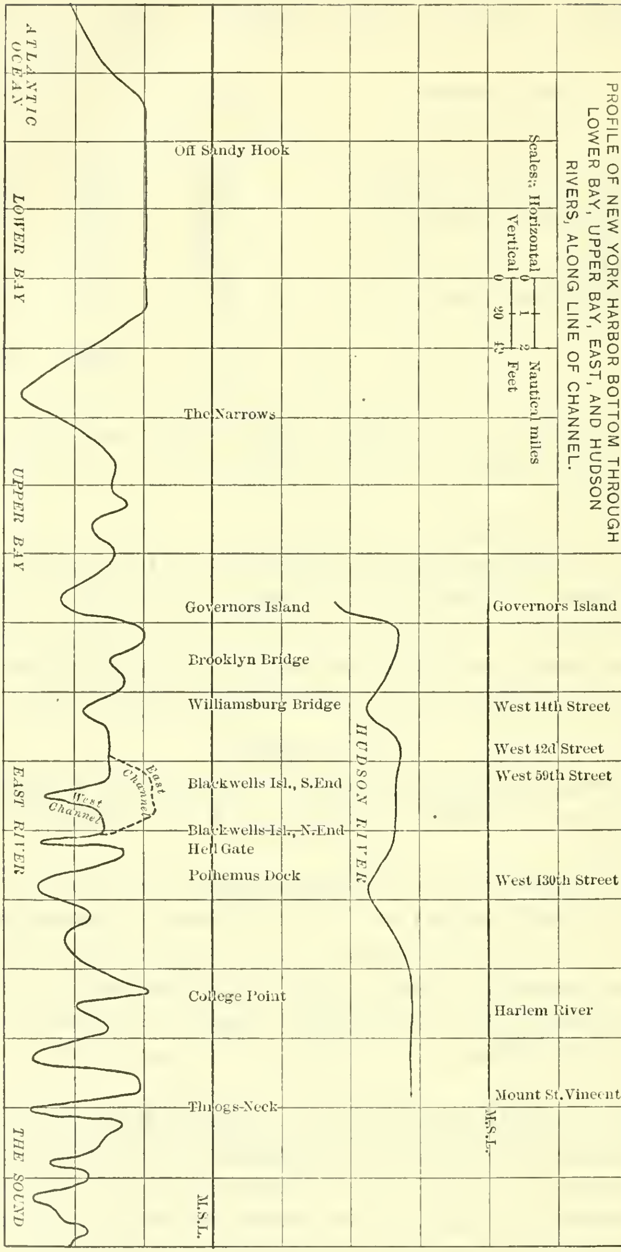

PROFILE OF NEW YORK HARBOR BOTTOM THROUGH LOWER BAY, UPPER BAY, EAST, AND HUDSON RIVERS, ALONG LINE OF CHANNEL.
Scales: Horizontal 0 1 2 Nautical miles
Vertical 0 20 40 Feet
ATLANTIC OCEAN
LOWER BAY
UPPER BAY
EAST RIVER
THE SOUND
HUDSON RIVER
Off Sandy Hook
The Narrows
Governors Island
Brooklyn Bridge
Williamsburg Bridge
Blackwells Isl., S.End
Blackwells Isl., N.End
Hell Gate
Polhemus Dock
College Point
Throgs Neck
M.S.L.
East Channel
West Channel
Governors Island
West 14th Street
West 42d Street
West 59th Street
West 130th Street
Harlem River
Mount St. Vincent
M.S.L.

FIG. 5.

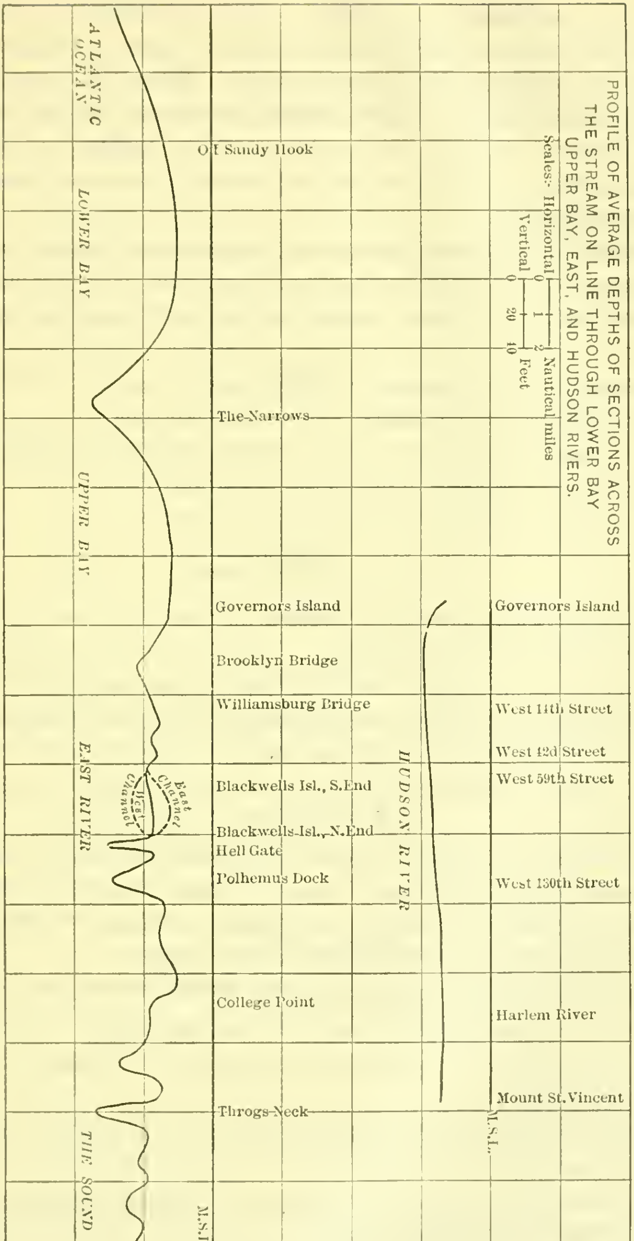

PROFILE OF AVERAGE DEPTHS OF SECTIONS ACROSS THE STREAM ON LINE THROUGH LOWER BAY UPPER BAY, EAST, AND HUDSON RIVERS.
Scales: Horizontal 0 1 2 Nautical miles
Vertical 0 20 40 Feet
ATLANTIC OCEAN
LOWER BAY
UPPER BAY
EAST RIVER
THE SOUND
HUDSON RIVER
West Channel
East Channel
O
Sandy Hook
The Narrows
Governors Island
Brooklyn Bridge
Williamsburg Bridge
Blackwells Isl., S.End
Blackwells Isl., N.End
Hell Gate
Polhemus Dock
College Point
Throgs Neck
M.S.L.
Governors Island
West 11th Street
West 42d Street
West 59th Street
West 130th Street
Harlem River
Mount St. Vincent
M.S.L.

FIG. 6.

When a combination of progressive wave and hydraulic conditions obtains, the greatest flood and ebb velocities occur irregularly, depending on which condition predominates. When the wave is restricted by interference or other causes, the curves (having ordinates which represent velocities or heights of water surface at different periods of time) depart from true sine curves, in amounts depending on the restriction.

The time of a complete tidal period, comprising a flood and an ebb, is 12 lunar hours. Under normal conditions, the flood and ebb flow during 6 lunar hours each, except for the small time lost at slack waters.

A lunar hour is equivalent to 1.03505 solar hours, or 6 lunar hours are equivalent to about 6 hours, 12.6 min., solar time.

In a channel where the current follows closely a true tidal wave action, which is harmonic in character, a floating object will be carried, during a complete flood or ebb period, a distance of

$$\frac{2}{\pi} \times A \times \frac{12.4206}{2} \text{ knots,}$$

or nearly $4A$ knots, in which A denotes the maximum velocity of the current, in knots per solar hour.

Similarly, the velocity at which a floating particle will be carried by the current at any time can be estimated from the formula,

$$A \cos. 30t,$$

where t denotes the number of lunar hours after the time of strength; or

$$A \cos. 28.98t,$$

where t denotes the number of solar hours after the time of strength.

By computing and plotting a number of these velocities at different stations in a selected locality, it is possible to make an estimate of how a floating object will be drifted by the currents.

There is some difference, however, due to the following causes:

1.—The under-run at the times of tidal changes;

2.—The inertia of the flowing stream when it changes from ebb to flood;

3.—The shore currents not being the same as the channel currents;

4.—Variations due to wind;

5.—Variations due to land-water flow coming down the rivers from drainage areas above;

6.—Variations in tidal ranges.

Owing to variations in channel sections and directions, which cause side currents, it is not safe to predict the course which a floating particle will take. It is safer to rely on observations with floats, many of which have been made by the Metropolitan Sewerage Commission,* and will be referred to later.

It must be remembered that the surface velocities do not readily indicate the mean velocities of tidal streams. Although the surface particles may have a known velocity in one direction, the velocity and direction of the particles beneath the surface often differ materially.

To determine accurately the mean velocity of a tidal current, it would be necessary to take synchronous observations at many points across a stream and at many points in depth below the surface. Such observations should extend over a considerable period of time in order to obtain fair averages.

Interference Tides.

When separate tidal waves enter a strait from opposite ends, there results in the strait an "interference" tide. This is not necessarily the algebraic sum of the two separate tides. In other words, when the two waves meet and overlap, the result of the overlap is not necessarily a superposition of one wave upon the other.

The actual tidal rise or fall above or below mean sea level, other things being equal and not acting as disturbing factors, at any hour of the tidal interval, is equal to a coefficient multiplied by the algebraic sum of the rises or falls of the two tides.† The difference in times between high and low water at each end of the strait is an important factor.

Interference tides occur in the harbor because separate tidal waves enter, one from the sea past Sandy Hook and the other from Long Island Sound past Willets Point. These separate tidal waves are not synchronous, that is, their periods of high or low water do not occur at the same time.

The Sound tide‡ (through Hell Gate) is distinctly traced to Governors Island, where, although much reduced in range, it is frequently found to affect the observed tide, mostly in the way of reducing the

* Report, April 30th, 1910.

† Henry Mitchell, Chief Physical Hydrographer, used 0.65 for the coefficient in computing the composite tide in the East River. Thus, the predicted tide was estimated by taking 0.65 times the algebraic sum of the difference in heights of the tides measured from mean sea level. U. S. Coast and Geodetic Survey, 1867. Appendix No. 13.

‡ U. S. Coast and Geodetic Survey, 1866. Appendix No. 6.

rise and fall. The southern tide from The Narrows meets a small propagation of the Sound tide, the lunar interval of which differs nearly 6 hours; the result is a reduction in range. This is the principal reason why the tidal range at Governors Island is less than that at Sandy Hook. The tidal range is also reduced by the expansion of The Narrows' tidal wave in Upper New York Bay.*

The East River is a strait, at one end of which the tidal range is 4.4 ft., and at the other 7.2 ft. Where the high range occurs (Willets Point) the tidal wave enters 3 hours, 5 min., later than the tidal wave at the other end (Governors Island). The two tides meet and cross or overlap each other at Hell Gate; and as they differ in times and heights, they cause contrasts of water elevations which call into existence violent currents.

The volume of water flowing through the strait in one direction is not necessarily equal to that flowing in the opposite direction, but depends on the relative times when each wave enters its respective end of the strait, as well as on the total range at each end.

If one wave enters later than the other, and has a greater range, the mean height of the water when the current is flowing one way will be higher than the mean height when the current is flowing contrariwise. Consequently, the sectional area of the stream in one case will be greater than in the other, and a larger volume of water is likely to pass during a tidal variation.

Suppose curves for the waves entering at each end of a strait be drawn, and then that the two curves be superimposed so that the distance between the peaks will represent the difference in time between high waters at each end, as shown in Fig. 7. The upper curves are true sinusoids, representing the mean ranges of tide at each end of the East River, namely Governors Island and Willets Point. The lower curves show the East River conditions for one set of observations, as recorded in the Report of the U. S. Coast and Geodetic Survey for 1888, Appendix No. 9.

Lunes† will be formed at *A* and *B*. When the curves are true sinusoids, their areas are equal; but, under actual tidal conditions, their areas are only approximately equal. These lunes are unbalanced, for the center of one is higher than that of the other, as shown by the crosses on Fig. 7.

* U. S. Coast and Geodetic Survey, 1867, Appendix No. 13.
† U. S. Coast and Geodetic Survey, 1886, Appendix No. 13.

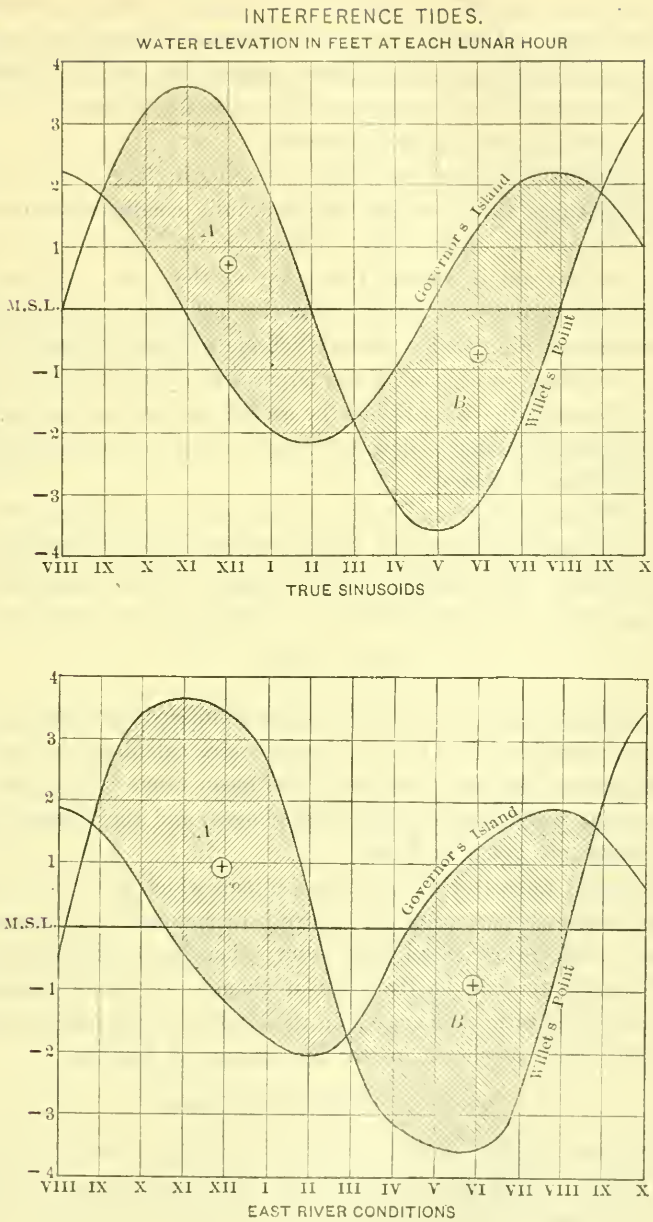

INTERFERENCE TIDES.
WATER ELEVATION IN FEET AT EACH LUNAR HOUR
4
3
2
1
M.S.L.
−1
−2
−3
−4
A
B
Governor's Island
Willet's Point
VIII IX X XI XII I II III IV V VI VII VIII IX X
TRUE SINUSOIDS
4
3
2
1
M.S.L.
−1
−2
−3
−4
A
B
Governor's Island
Willet's Point
VIII IX X XI XII I II III IV V VI VII VIII IX X
EAST RIVER CONDITIONS

FIG. 7.

During the time of the formation of Lune *A*, the water surface at Willets Point is higher than that at Governors Island, thus tending to produce a current from the former toward the latter. Conversely, during the time of the formation of Lune *B* the tendency is to produce a current in the opposite direction.

The centers of these lunes are at the mean levels of the water during the times the water has the hydraulic conditions represented by Lunes *A* and *B*, respectively. Therefore, the mean cross-sectional area of one stream is greater than that of the other. If the mean velocities of the currents are the same in each direction then, as the hydraulic slopes are equal, a greater volume will flow in one direction than in the other, on account of the larger area of the stream section.

In the case of the East River, the greater section and the tendency toward a greater volume of flow is for the current running from Willets Point to Governors Island.

The Harlem River conditions are the result of interference tides, with waves entering from the Hudson River and the East River at different times. The Harlem River is really a strait, and what has been said about the East River would also apply in a modified way to it.

Tidal Prisms.

A tidal prism is the volume of water contained between low-water and high-water levels. The tidal prisms were calculated by multiplying the area of the water surface by the mean range of the tide. The areas of the water surfaces, the tidal ranges, and the volumes of the tidal prisms are given in Table 2.

For equal ranges of tide, the percentage which the tidal prism is of the volume below mean low water will increase inversely as the average depth below mean low water; therefore, the shallower the water the greater will be the ratio of change of volume during each tide. The least ratio of change is in the Hudson River, and the greatest in Newark Bay, as shown in the last column of Table 2.

Quantity of Water in Harbor.

The water surface of the harbor was calculated from the Government charts.* This surface is 65.12 sq. miles, exclusive of the Lower Bay and Jamaica Bay. The areas of the several divisions are given

* The boundaries of the harbor divisions are given in Appendix A.

in Table 2. The area of the Lower Bay is 124.3 sq. miles, and of Jamaica Bay 21.61 sq. miles.

TABLE 2.—AREAS, DEPTHS, TIDAL RANGES, VOLUMES, AND TIDAL PRISMS.

Part of harbor.	Area, in square miles.	Average depth, in feet.	Mean tidal range, in feet.	Volume below M. L. W., in millions of cubic feet.	Tidal prism, in millions of cubic feet.	Percentage.*
Upper Bay	20.74	22.4	4.4	12 970	2 541	19.6
Hudson River, Battery to Mt. St. Vincent.	14.49	30.7	4.2	12 330	1 697	13.7
East River, Battery to East 88th Street	3.50	31.7	4.7	3 091	459	14.8
East River, East 88th Street to Old Ferry Point	9.12	22.3	6.2	5 680	1 575	27.7
East River, Old Ferry Point to Throgs Neck	2.58	38.8	7.1	2 791	511	18.3
Harlem River	0.59	11.3	5.3	189	88.2	46.6
Newark Bay	8.35	6.6	4.6	1 542	1 071	69.5
Kill van Kull	1.12	23.4	4.8	728	149.8	20.6
Arthur Kill	4.63	13.1	5.4	1 690	697	41.2
Totals	65.12	....	...	41 011	8 789.0	....

* Ratio of tidal prism to volumes below M. L. W. expressed as a percentage.

An estimate of the volumes of water below mean low water was made in the following manner: For the Upper Bay, Newark Bay, and the East River between East 88th Street, Manhattan, and Throgs Neck, the chart areas were divided into squares, and the depths as given on the chart were averaged for each square or fraction thereof. The shore lines were taken as marked on the charts for mean low water. The volumes under each square were found and added. For the Hudson River, East River south of 88th Street, Harlem River, Arthur Kill, and Kill van Kull, cross-sections were drawn at regular intervals, from information on Government charts of large scale, and cross-sectional areas were obtained. From these areas and their distances apart the volumes were calculated. The results are given in Table 2.

At any hour, there is more water in the harbor than is given by the sum of these figures, because mean low water does not occur at the same hour in all its parts.

The volume of water is variable from hour to hour, and the approximate mean value is nearly the sum of the quantity of water below mean low water and one-half of the mean tidal prism. The tidal prisms are given in Table 2. The average quantity of water, therefore, is not far from the sum of 41 011 000 000 and 4 394 500 000, or 45 405 500 000 cu. ft., exclusive of the Lower Bay and Jamaica Bay.

Tidal Ranges.

The tidal ranges, or the rise and fall of the tides, are not the same at all points in the harbor; also, the hours at which high and low water occur at different points vary considerably. These statistics are given in Table 3.

TABLE 3.—Mean Range of Tides, and Times of High and Low Water.*

Station.	Mean tidal range, in feet	Times.†		Differences in Times.‡	
		H. W.	L. W.	H. W.	L. W.
Sandy Hook (Horseshoe)	4.7	7.35	1.27	—0.29	—0.38
Canarsie, Jamaica Bay	4.2	8.34	2.35	0.30	0.30
Tottenville, Arthur Kill	5.6	7.55	1.59	—0.09	—0.06
Shooters Island, Newark Bay	4.6	8.20	2.28	0.17	0.23
Passaic Light, Newark Bay	4.7	8.41	2.59	0.38	0.54
Fort Hamilton, Narrows	4.6	7.41	1.38	—0.23	—0.28
Governors Island	4.4	8.04	2.05	0.00	0.00
Blackwells Island Light	5.3	9.54	3.39	1.50	1.33
Throgs Neck	7.3	11.09	5.14	3.05	3.09
Spuyten Duyvil	4.0	8.49	2.51	0.45	0.46

* Tide Tables, U. S. Coast and Geodetic Survey.

† Solar time, in hours and minutes, after transit of moon.

‡ From that at Governors Island, in hours and minutes.

Current Velocities.

The current velocities are variable throughout the harbor, and are dependent on the conditions existing at the time of observation. Under normal conditions, the coefficient which expresses the relation between the swiftest surface thread of a current and the mean cross-sectional velocity is probably fairly constant for any one section, although it is affected by inertia and the reversal of the currents due to change of direction, which reversal does not take place simultaneously throughout a section.

Under normal conditions, the swiftest surface velocities (or the velocities just under the surface) of the tidal currents, when flowing at strength, are given in Table 4, in knots per solar hour, as well as the normal duration of surface currents, in lunar hours. These velocities and durations were obtained from records of the Coast and Geodetic Survey and from float observations made by the Metropolitan Sewerage Commission.

The mean surface velocity of the swiftest thread is approximately $\frac{2}{\pi}$ times the velocity at strength, as the wave formation is harmonic in character.

TABLE 4.—NORMAL SWIFTEST SURFACE VELOCITIES AT STRENGTH, IN KNOTS; AND NORMAL DURATION OF SURFACE CURRENTS, IN LUNAR HOURS.

Station.	Velocity at Strength.		Duration.* in Lunar Hours.	
	Ebb.	Flood.	Ebb.	Flood.
The Narrows	2.1	1.8	6 7	5.3
Hudson River, off 39th Street	3.0	2 0	6 7	5.3
East River, Brooklyn Bridge	3.8	3.6	6.6	5 4
" " off 11th Street	3 0	2.9	6 1	5.9
" " off 31st Street	2.9	2.6	6 0	6.0
" " at Hell Gate	4.8	4.7	5 9	6.1
" " off Old Ferry Point	1.3	1.3	5.8	6.2
Kill van Kull, off Port Richmond	2.2	1.9	6.3	5.7
" " " 0.1 mile S. of Bergen Point	2.0	1.8	6.4	5.6
Harlem River, off 114th Street	1.0	1.0	5.9	6.1
" " 600 ft. N. of High Bridge	1.9	1.8	6.0	6.0

* The durations of slack-water are not the same in all parts of the harbor, nor does the current change at the same time in all parts of a section. The slack-water periods are included in the duration of the currents, which add to 12 lunar hours.

Some observations of the velocities of the currents at different depths have been made in a few sections in the harbor.* They were made at one or at only a few stations in a section, and the records are not sufficiently full to be of material value in determining the mean cross-sectional velocity, or the value of the coefficient expressing the relation between the velocity of the swiftest surface thread and the mean velocity of the section. They show, however, that the currents do not reverse simultaneously in a section, and also that the period of time occupied by a complete change of current is not the same for ebb to flood as for flood to ebb. Generally speaking, the period is longer in the former case and shorter in the latter.

The surface of some parts of the bottom of the harbor is much freer from fine silt and light deposits than others. It also appears that when the velocities exceed 1½ ft. per sec., the finer and lighter materials are washed away and deposited in the deeper or quieter parts.

* Reports of U. S. Coast and Geodetic Survey.

Effect of Tidal Range on Velocity.

In the East River the tidal currents are nearly hydraulic, that is, they flow from the body having temporarily the higher water-surface level to the one having temporarily the lower. In other words, the flow is caused by the difference in height which temporarily exists between the bodies connected. In consequence, the velocities in the East River vary closely as the square root of the range of tide.

In the Hudson River the tidal currents are due chiefly to the progressive wave motion, as is shown by the fact that the greatest flood and ebb velocities occur at nearly the times of local high and low waters. In the Hudson, therefore, the velocities vary directly as the range of tide.

In the Kill van Kull and in the Arthur Kill the tidal currents are nearly hydraulic, and in them, as in the East River, the velocities vary closely as the square root of the range of tide.

In The Narrows the tidal currents are partly hydraulic and partly due to the progressive wave motion. In consequence, the velocities there vary approximately midway between the square root of and directly as the range of tide.

In the Harlem River the tidal currents are nearly hydraulic, and are due to a temporary difference in water level in the East River and in the Hudson. Therefore, the velocities vary approximately as the square root of the range of tide.

In Rockaway Inlet, joining Jamaica Bay and the Atlantic Ocean, the currents are hydraulic. The velocities, therefore, vary as the square root of the range of tide, the greatest velocities occurring about 3 hours before high or low water.

Currents at Each Lunar Hour.

The currents which exist in the harbor at each lunar hour of a tidal cycle are given below. The directions of the currents were obtained chiefly from the float experiments of the Metropolitan Sewerage Commission, and were checked from the curves shown on Fig. 8. Other observations were also made, for purposes of verification. The construction of the curves in Fig. 8 is explained on page 730.

I Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; into the Upper Bay, through the Kill

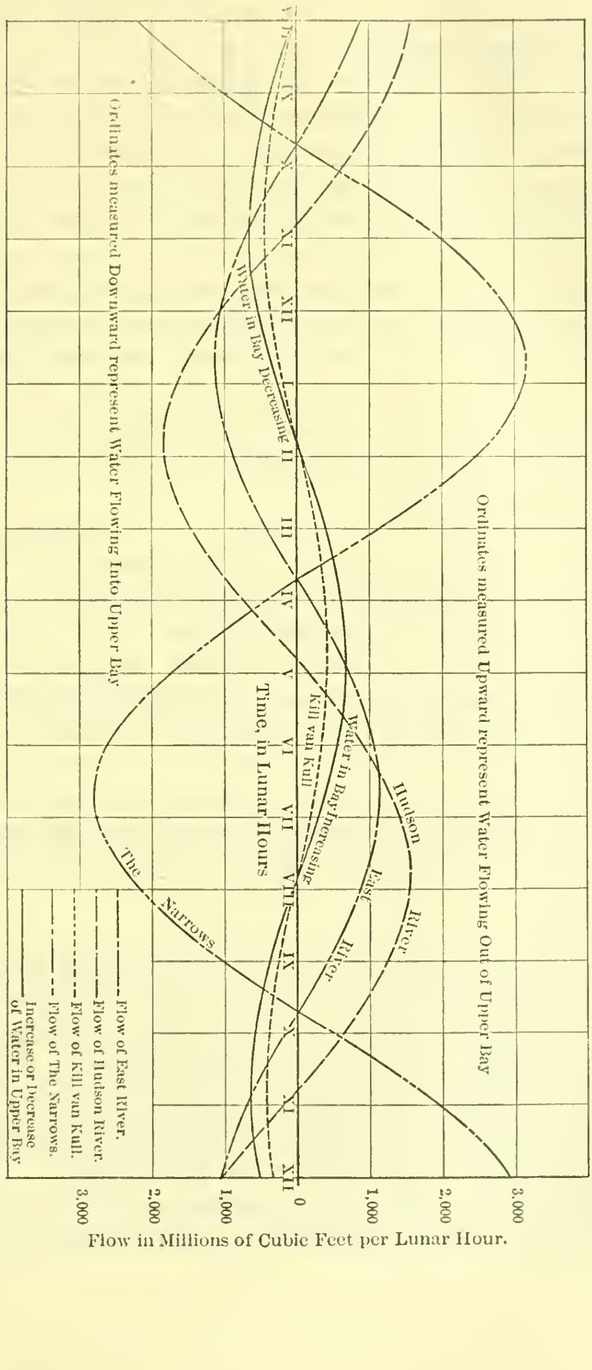

Ordinates measured Downward represent Water Flowing Into Upper Bay
Ordinates measured Upward represent Water Flowing Out of Upper Bay
Time, in Lunar Hours
Water in Bay Decreasing
Water in Bay Increasing
Kill van Kull
Hudson River
East River
The Narrows
IX
X
XI
XII
I
II
III
IV
V
VI
VII
VIII
IX
X
XI
XII
Flow of East River.
Flow of Hudson River.
Flow of Kill van Kull.
Flow of The Narrows.
Increase or Decrease of Water in Upper Bay
3,000
2,000
1,000
0
1,000
2,000
3,000
Flow in Millions of Cubic Feet per Lunar Hour.

FIG. 8.

van Kull, the East River, and the Hudson River; and the water level in the Upper Bay is falling.

II Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; into the Upper Bay through the East River and the Hudson River; the Kill van Kull is nearly slack; and the water in the Upper Bay is at about mean low water.

III Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; out of the Upper Bay, through the Kill van Kull; into the Upper Bay, through the East River and the Hudson River; and the water in the Upper Bay is rising.

IV Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; into the Upper Bay, from the Hudson River; out of the Upper Bay and Hudson River, into the East River, out of the Upper Bay, into the Kill van Kull; and the water in the Bay is rising.

V Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; the Hudson River is nearly slack; the water is flowing out of the Upper Bay, through the East River and the Kill van Kull; and the water in the Bay is rising.

VI Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; out of the Upper Bay, through the Hudson River, East River, and Kill van Kull; and the water in the Bay is rising.

VII Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; out of the Upper Bay, through the Hudson River, East River, and Kill van Kull; and the water in the Bay is rising.

VIII Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; out of the Upper Bay, through the East River and the Hudson River; the Kill van Kull is nearly slack; and the water in the Bay is at about mean high water.

IX Lunar Hour.—The water is flowing into the Upper Bay, through The Narrows, from the sea; into the Upper Bay, through the Kill van Kull; out of the Upper Bay, through the Hudson River and the East River; and the water in the Bay is falling.

X Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; out of the Upper Bay, through

the Hudson River; into the Upper Bay and Hudson River, through the East River; into the Upper Bay, through the Kill van Kull; and the water in the Bay is falling.

XI Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; the Hudson River is nearly slack; the water is flowing into the Upper Bay, through the East River and the Kill van Kull; and the water in the Bay is falling.

XII Lunar Hour.—The water is flowing out of the Upper Bay, through The Narrows, toward the sea; into the Upper Bay, through the Hudson River, East River, and Kill van Kull; and the water in the Bay is falling.

Water Levels at Each Lunar Hour.

The water surfaces are changing continually throughout the tidal cycle, and the differences in elevation create slopes which are influential in causing the water to flow first in one, and then in a reverse, direction.

Figs. 9 to 20, inclusive, show the water surfaces at each lunar hour during the cycle. By comparing one figure with those for the preceding and subsequent hours, the rapid changes in levels are apparent. The figures show the cause of the violent currents through Hell Gate, and the steep slopes which exist at this interesting place. These figures were drawn from information contained in Appendix 15, Report of 1887, and Appendix 9, Report of 1888, of the U. S. Coast and Geodetic Survey, with the elevations for some additional points calculated by the writer from Tide Table information.

Land-Water Discharges.

The land-water discharged into the harbor comes from the rivers and their contributing basins. The U. S. Coast and Geodetic Survey made an estimate* of this land-water, based on the records of the gauging stations maintained by the U. S. Geological Survey and the New York State Engineer and Surveyor at Mechanicsville on the Hudson (1890 to 1905), and at a point about 4 miles below Rexford Flats on the Mohawk River (1901-1905). The quantity of land-water running off areas below these stations has been assumed to be related to these

* Letter from Survey to Metropolitan Sewerage Commission of New York, August 14th, 1908.

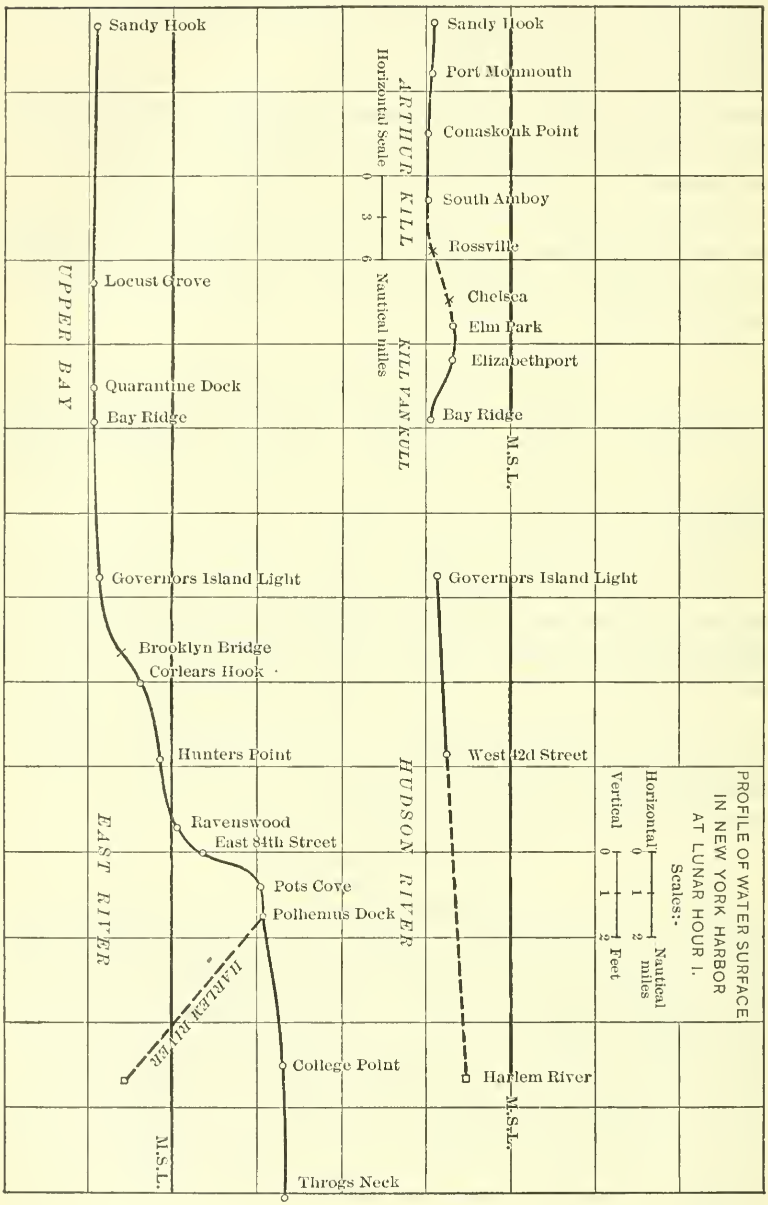

Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
HARLEM RIVER
UPPER BAY
EAST RIVER
M.S.L.
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
Horizontal Scale
0 3 6
Nautical miles
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
M.S.L.
Governors Island Light
West 42d Street
Harlem River
M.S.L.
PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR I.
Scales:-
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet

FIG. 9.

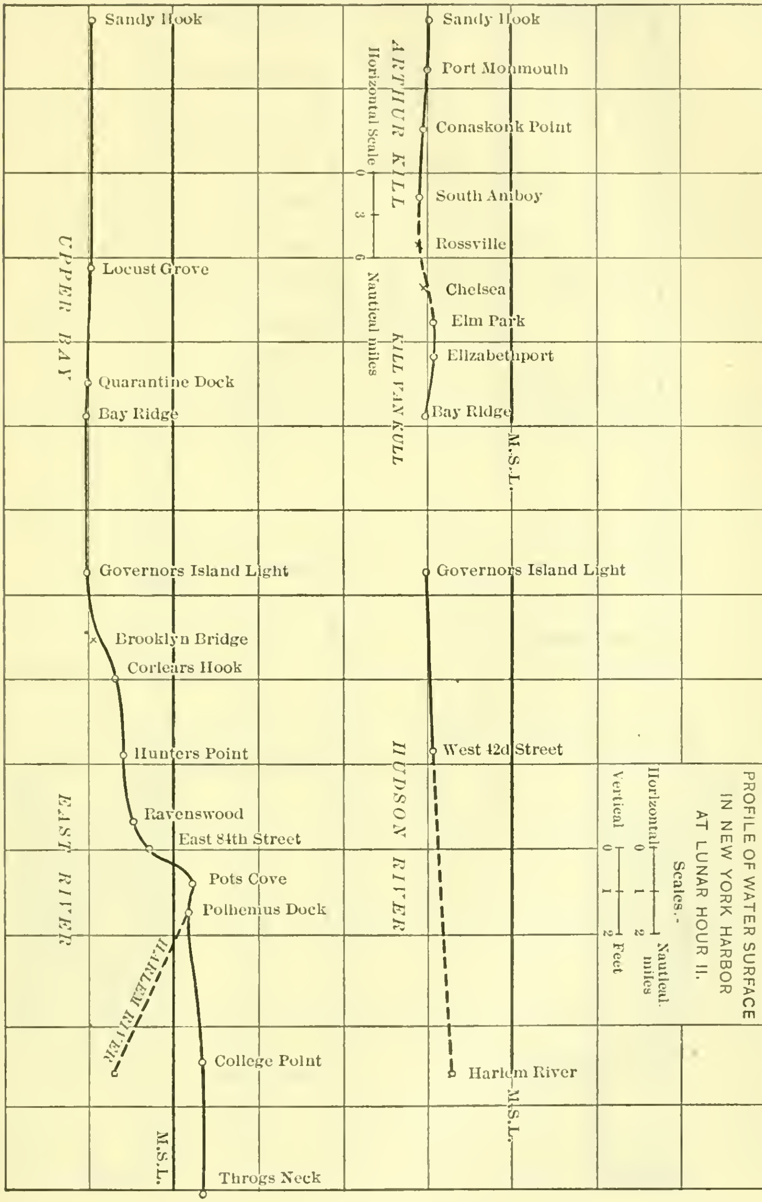

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR II.
Scales.-
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
UPPER BAY
EAST RIVER
HARLEM RIVER
M.S.L.
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
Governors Island Light
West 42d Street
Harlem River
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
Horizontal Scale 0 3 6 Nautical miles
M.S.L.

FIG. 10.

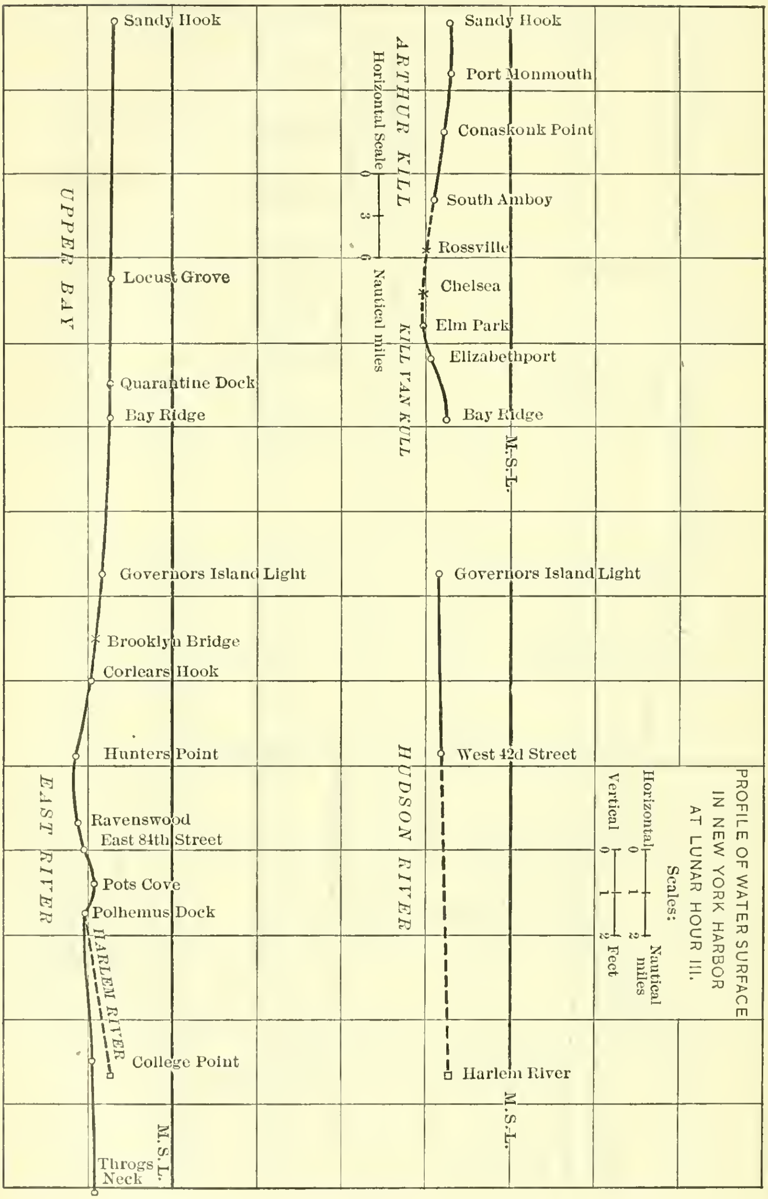
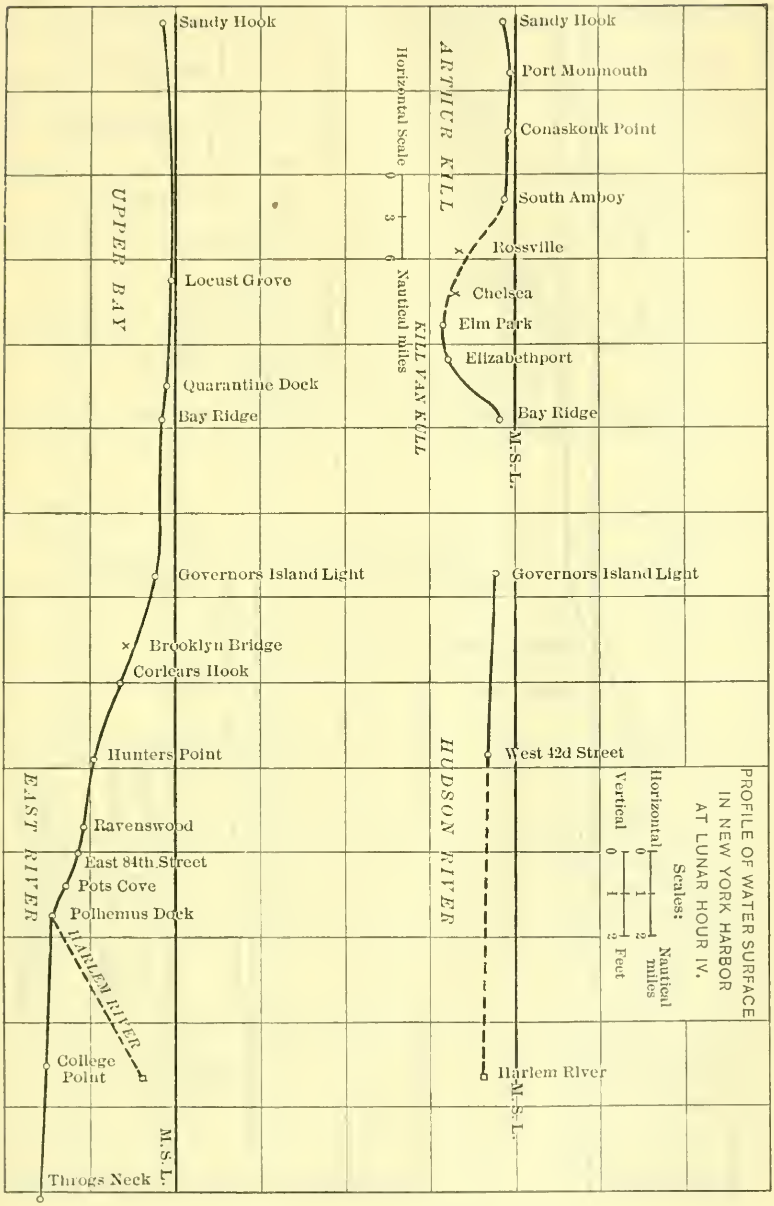

UPPER BAY
EAST RIVER
HARLEM RIVER
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
M.S.L.
ARTHUR KILL
KILL VAN KULL
Horizontal Scale
0 3 6
Nautical miles
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
M.S.L.
HUDSON RIVER
Governors Island Light
West 42d Street
Harlem River
M.S.L.
PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR III.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet

Fig. 11.



UPPER BAY
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
EAST RIVER
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
HARLEM RIVER
College Point
Throgs Neck
M. S. L.
Horizontal Scale
0
3
6
Nautical miles
ARTHUR KILL
KILL VAN KULL
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
M. S. L.
Governors Island Light
HUDSON RIVER
West 42d Street
Harlem River
M. S. L.
PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR IV.
Scales:
Horizontal
0
1
2
Nautical miles
Vertical
0
1
2
Feet

Fig. 12.

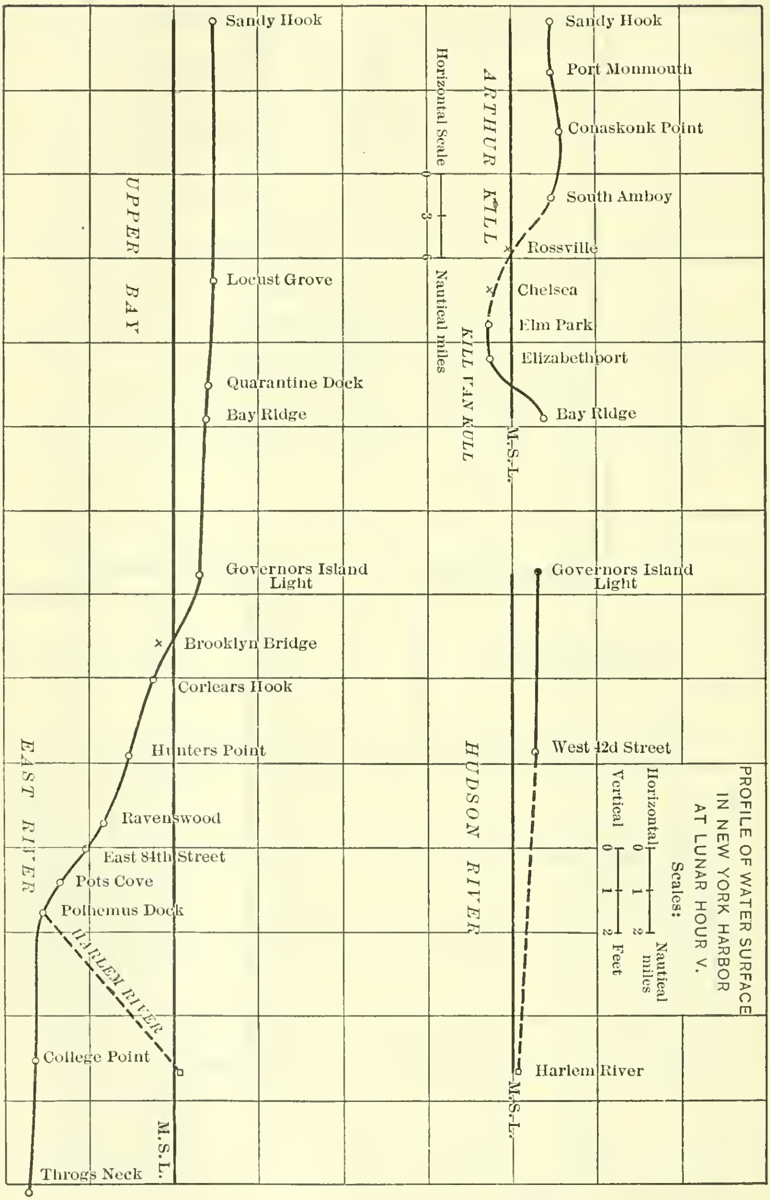

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR V.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
Horizontal Scale 0 3 6 Nautical miles
UPPER BAY
EAST RIVER
HARLEM RIVER
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
West 42d Street
Harlem River

FIG. 13.

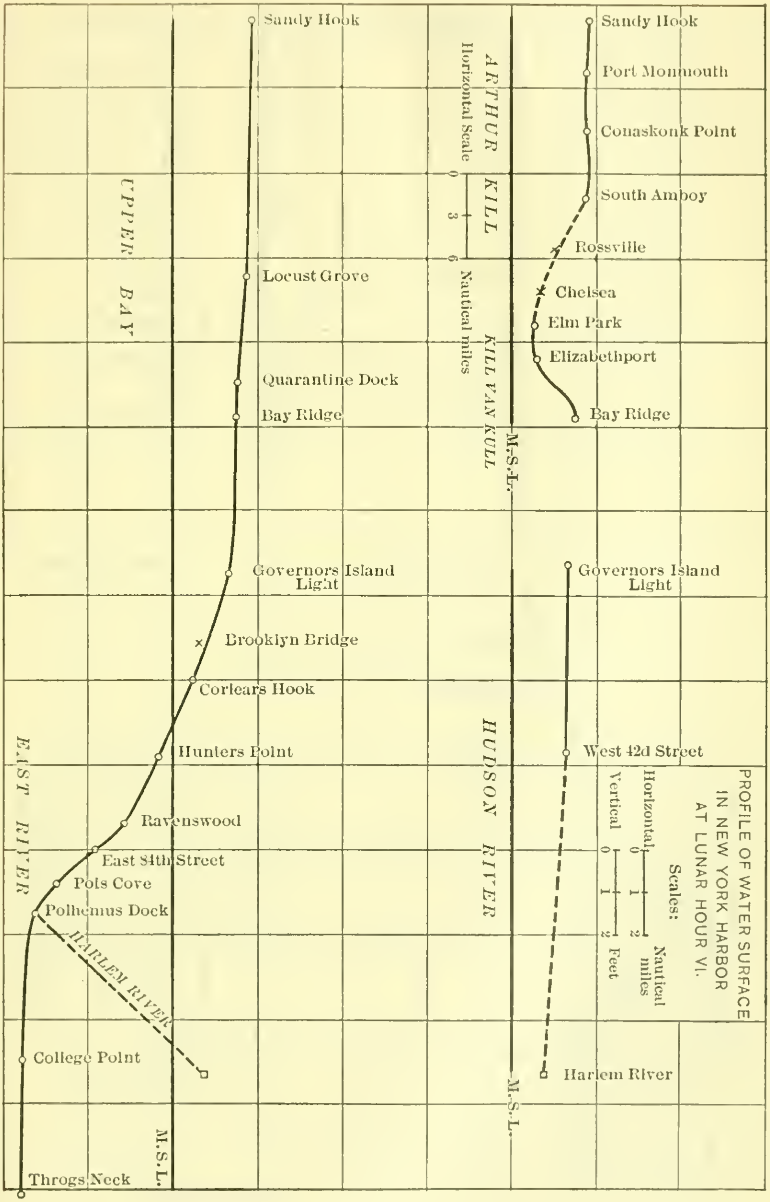

Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pols Cove
Polhemus Dock
College Point
Throgs Neck
UPPER BAY
EAST RIVER
HARLEM RIVER
M.S.L.
ARTHUR KILL
KILL VAN KULL
Horizontal Scale
0 3 6 Nautical miles
HUDSON RIVER
M.S.L.
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
Governors Island Light
West 42d Street
Harlem River
PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR VI.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet

FIG. 14.

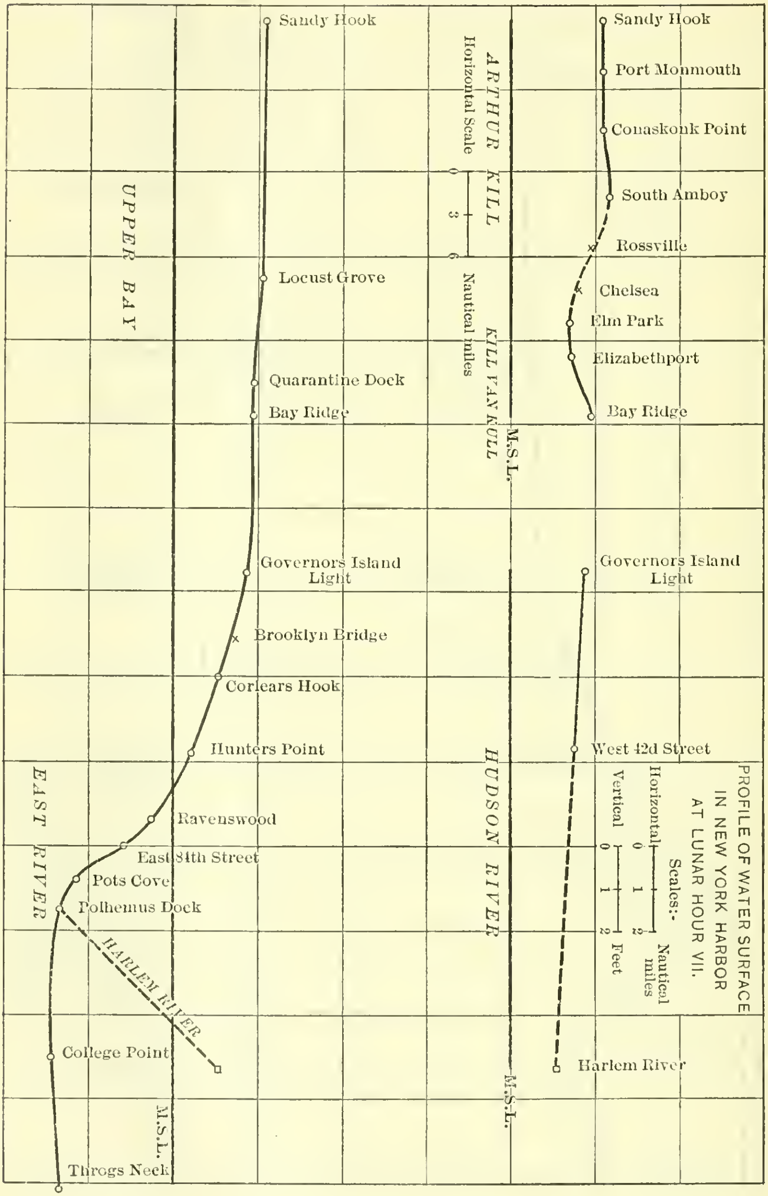
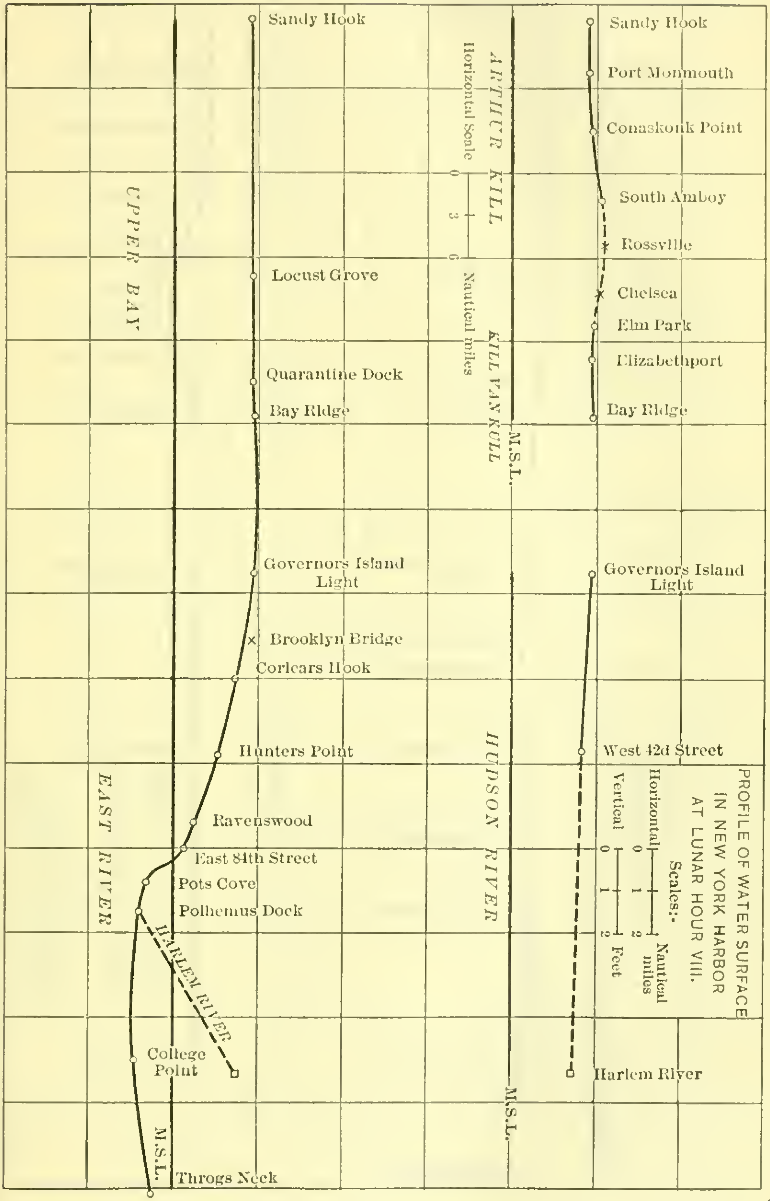

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR VII.
Scales:-
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
UPPER BAY
EAST RIVER
HARLEM RIVER
ARTHUR KILL
Horizontal Scale 0 3 6 Nautical miles
KILL VAN KULL
HUDSON RIVER
M.S.L.
M.S.L.
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
Governors Island Light
West 42d Street
Harlem River

FIG. 15.



PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR VIII.
Scales:-
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
ARTHUR KILL
KILL VAN KULL
Horizontal Scale 0 3 6 Nautical miles
UPPER BAY
EAST RIVER
HUDSON RIVER
HARLEM RIVER
M.S.L.
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
Locust Grove
Quarantine Dock
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
West 42d Street
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Harlem River
Throgs Neck

FIG. 16.

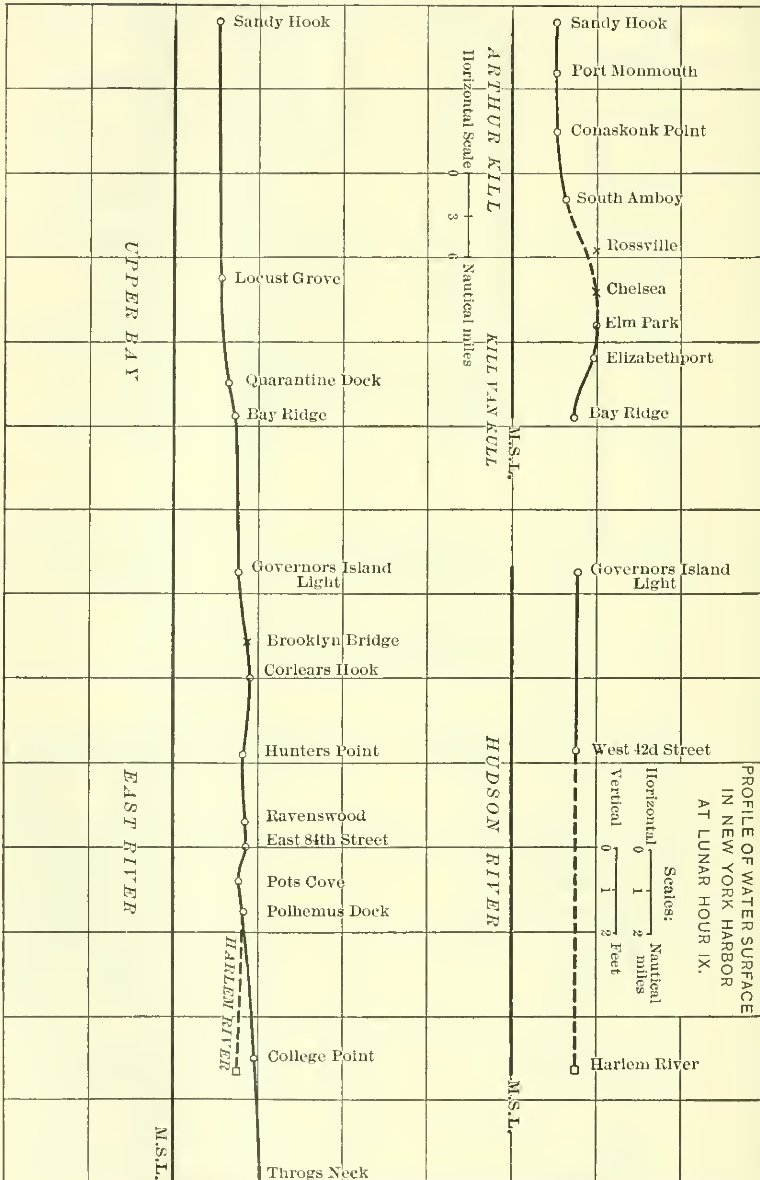
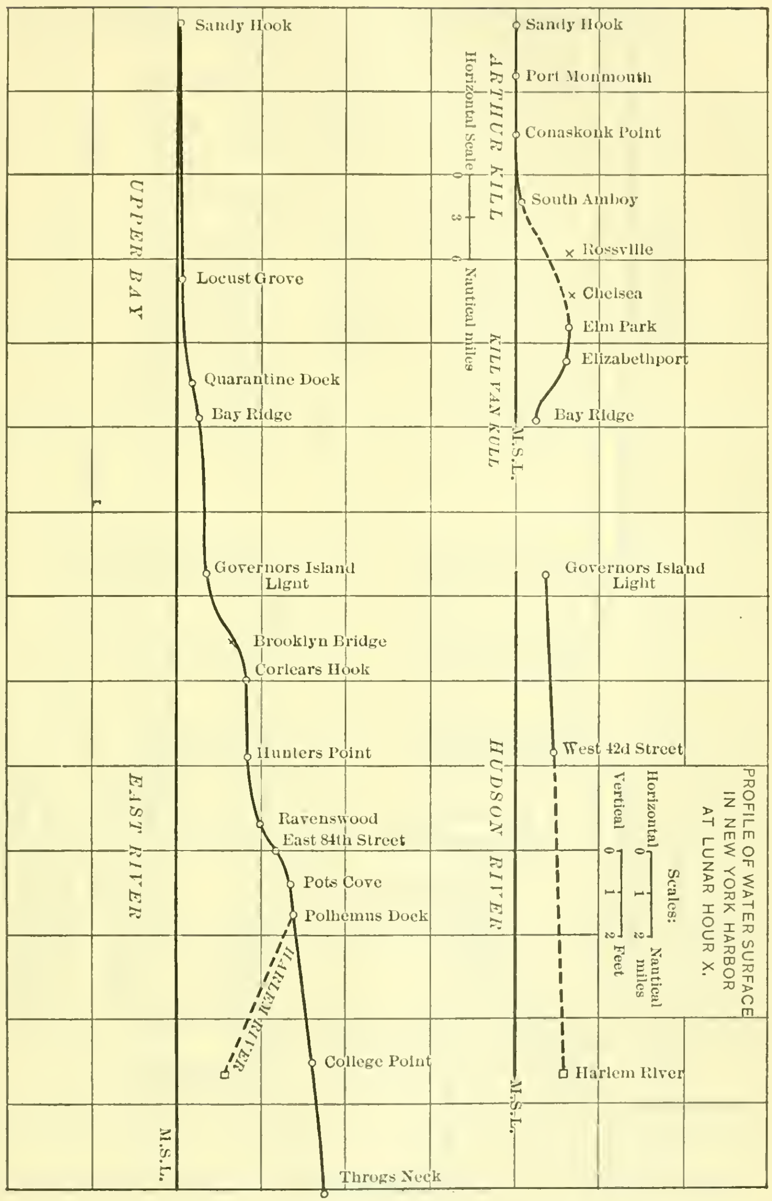

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR IX.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
UPPER BAY
EAST RIVER
HARLEM RIVER
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
ARTHUR KILL
Horizontal Scale 0 3 6 Nautical miles
KILL VAN KULL
HUDSON RIVER
M.S.L.
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
Governors Island Light
West 42d Street
Harlem River

Fig. 17.



PROFILE OF WATER SURFACE IN NEW YORK HARBOR AT LUNAR HOUR X.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
UPPER BAY
EAST RIVER
HARLEM RIVER
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
Horizontal Scale 0 3 6 Nautical miles
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
West 42d Street
Harlem River

Fig. 18.

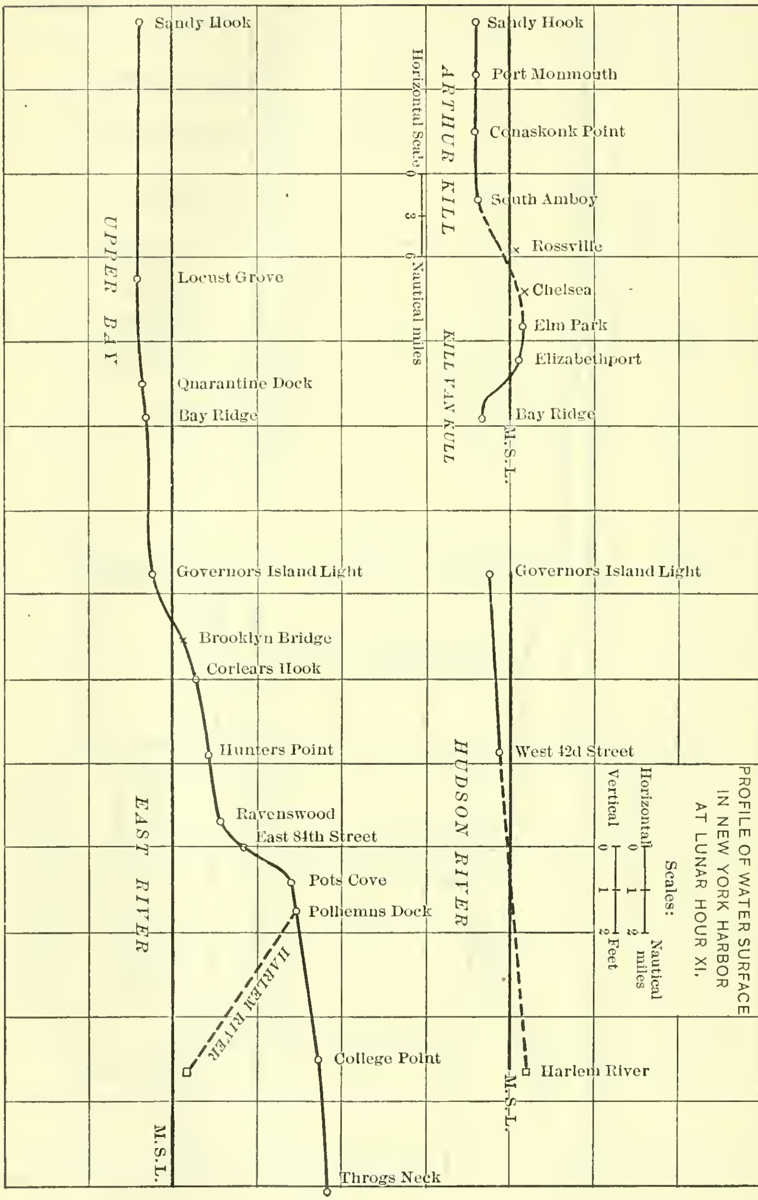

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR XI.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
Horizontal Scale 0 3 6 Nautical miles
UPPER BAY
EAST RIVER
HARLEM RIVER
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
M.S.L.
M.S.L.
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
Sandy Hook
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
Bay Ridge
Governors Island Light
West 42d Street
Harlem River

Fig. 19.

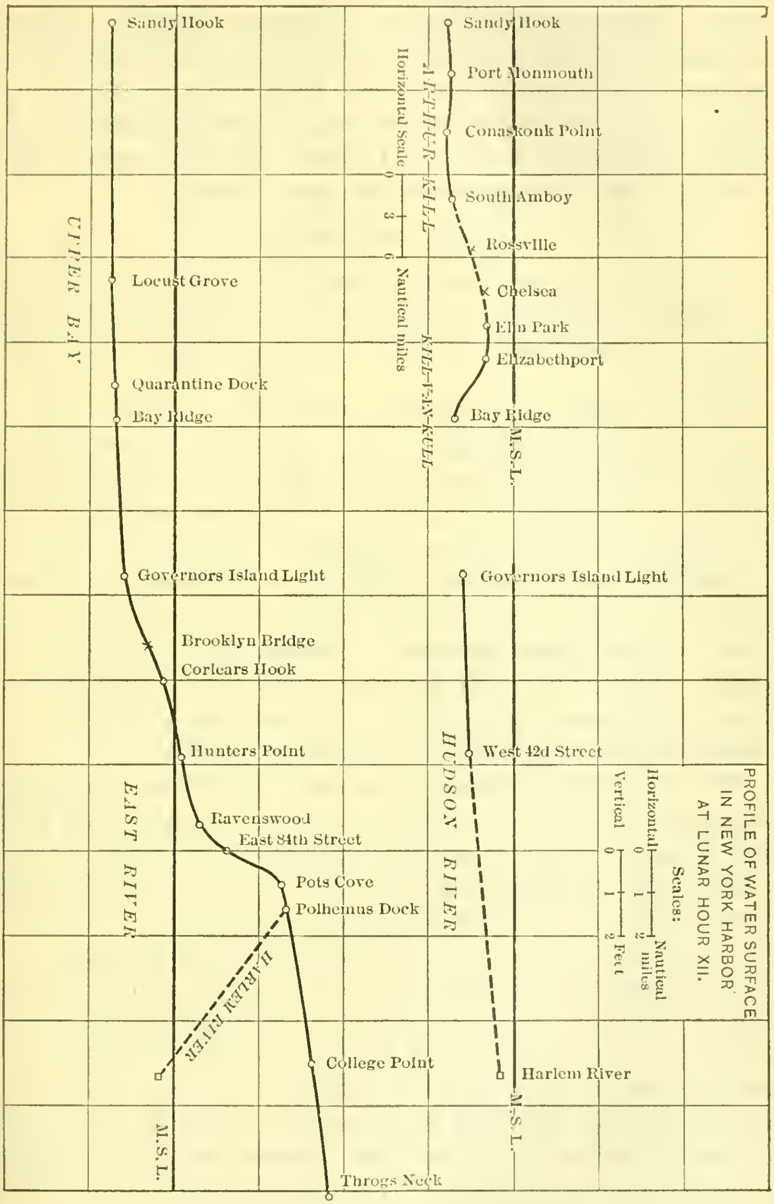

PROFILE OF WATER SURFACE
IN NEW YORK HARBOR
AT LUNAR HOUR XII.
Scales:
Horizontal 0 1 2 Nautical miles
Vertical 0 1 2 Feet
UPPER BAY
EAST RIVER
HARLEM RIVER
ARTHUR KILL
KILL VAN KULL
HUDSON RIVER
Horizontal Scale 0 3 6 Nautical miles
M.S.L.
Sandy Hook
Locust Grove
Quarantine Dock
Bay Ridge
Governors Island Light
Brooklyn Bridge
Corlears Hook
Hunters Point
Ravenswood
East 84th Street
Pots Cove
Polhemus Dock
College Point
Throgs Neck
Port Monmouth
Conaskonk Point
South Amboy
Rossville
Chelsea
Elm Park
Elizabethport
West 42d Street
Harlem River

Fig. 20.

areas as the measured discharges are to the areas above the gauging stations. The results thus obtained were then multiplied by factors representing the ratios of the annual rainfalls over the regions under consideration to the annual rainfall over the regions above the gauging stations. These rainfall ratios were obtained from Professor A. J. Henry's "Climatology of the United States." The final results or estimates of the land-water discharges are given in Table 5.

Current Phenomena.

The turn of the tide takes place first along the shore, so that the shore currents often flow contrariwise to the channel currents. In The Narrows, the first change of current appears on the east side of the channel.

When the tidal currents turn from flood to ebb or from ebb to flood, the inertia of the moving stream plays a very important part. As the specific gravity of the sea outside of New York bar is about 1.024, the sea-water is approximately 2½% heavier than the fresh-water discharged from the rivers. As the tide begins to flood, there is a tendency for this lighter fresh-water to flow out over the top of the incoming heavier salt-water, and for the formation of top and bottom currents flowing in opposite directions. Consequently, an "under-run" is formed. The "under-run" of salt-water up the Hudson River extends above Poughkeepsie, where the water is often found brackish during seasons when the river discharge of land-water is small.

The Metropolitan Sewerage Commission made more than ninety float experiments during 1907, 1908, and 1909. The floats were made deep, in order that they might not be influenced by the wind, and were of two kinds: a submerged can supported by a wire to a can float, and a wooden post carrying vanes at the lower end supported by a wooden float at the upper end. The can float was made in the following way: The submerged can was 6¼ in. in diameter and 14 in. high, and had a ring at the top to which was fastened a copper wire connecting it to the float proper. The length of this wire could be varied, but was usually about 4 ft. The float was a can, 5⅜ in. in diameter and 5 in. high, hermetically sealed, and had a socket on the cover in which could be placed a staff for a small flag. The submerged can had an opening in the cover through which sand could be introduced so as to weight the apparatus until the float became almost submerged.

TABLE 5.—AVERAGE DISCHARGE OF FRESH WATER, IN CUBIC FEET PER SIX LUNAR HOURS, IN NEW YORK HARBOR.

Contributing basin.	Area, in square miles.	Jan.	Feb.	Mar.	Apr.	May.	June.
Hudson River (above Battery Place)	13 369	405 467 000	317 536 000	1 149 373 000	1 190 980 000	615 645 000	500 014 000
East River, between Governors Island and Randalls Island	36	1 274 000	984 000	3 577 000	3 689 000	1 900 000	1 565 000
Newark Bay and Kill van Kull, ignoring discharge through Arthur Kill	1 139	39 348 000	30 808 000	111 561 000	115 608 000	59 738 000	48 515 000
Newark Bay and Kill van Kull, allowing 83.66313% for the proportion from Newark Bay flowing through the Kill van Kull	953	32 920 000	25 775 000	93 335 000	96 721 000	49 979 000	40 589 000
Upper Bay (exclusive of all above tributaries)	37	1 297 000	1 028 000	3 689 000	3 823 000	1 990 000	1 610 000
Total entering Upper Bay above The Narrows	14 395	440 958 000	345 323 000	1 249 974 000	1 295 213 000	669 514 000	543 778 000
Arthur Kill, allowing 16.33687% for proportion from Newark Bay	353	12 330 000	9 661 000	34 949 000	36 214 000	18 724 000	15 214 000
Raritan River	1 105	37 314 000	29 221 000	105 771 000	109 594 000	56 653 000	46 011 000
*Total entering Lower Bay, exclusive of that entering Upper Bay	1 458	49 644 000	38 882 000	140 720 000	145 808 000	75 377 000	61 225 000

Contributing basin.	Area, in square miles.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
Hudson River (above Battery Place)	13 369	320 734 000	286 371 000	325 652 000	453 042 000	419 216 000	539 027 000	543 588 000
East River, between Governors Island and Randalls Island	36	984 000	894 000	1 028 000	1 408 000	1 297 000	1 677 000	1 690 000
Newark Bay and Kill van Kull, ignoring discharge through Arthur Kill	1 139	31 143 000	27 790 000	31 613 000	43 976 000	40 667 000	52 315 000	52 757 000
Newark Bay and Kill van Kull, allowing 83.66313% for the proportion from Newark Bay flowing through the Kill van Kull	953	26 055 000	23 250 000	26 448 000	36 792 000	34 023 000	43 768 000	44 138 000
Upper Bay (exclusive of all above tributaries)	37	1 028 000	917 000	1 051 000	1 476 000	1 364 000	1 741 000	1 751 000
Total entering Upper Bay above The Narrows	14 395	348 801 000	311 432 000	354 179 000	492 718 000	455 900 000	586 216 000	591 167 000
Arthur Kill allowing 16.33687% for proportion from Newark Bay	353	9 761 000	8 698 000	9 905 000	13 757 000	12 725 000	16 394 000	16 528 000
Raritan River	1 105	29 511 000	26 337 000	29 958 000	41 673 000	38 588 000	49 610 000	50 002 000
*Total entering Lower Bay, exclusive of that entering Upper Bay	1 458	39 272 000	35 035 000	39 863 000	55 430 000	51 313 000	66 004 000	66 530 000

NOTE.—Of the water entering and leaving Newark Bay and tributaries, 83.66313% passes Kill van Kull and 16.33687% passes the upper end of Arthur Kill.

Annual rainfall:

Hudson and Mohawk Valleys, above gauging stations	37 in.	Newark Bay Basin	44 in.
Hudson Valley below gauging stations	41 "	Upper Bay	45 "
East River	44½ "	Arthur Kill Basin	45 "
		Raritan River Basin	43 "

*Total from Arthur Kill, Newark Bay, and Raritan River, but not Shrewsbury River or similar drainage areas.

The second float consisted of a 4 by 4-in. wooden standard, varying in length, but usually about 6 ft. long. At the lower end, four 18 by 18-in. vanes of No. 14 B. w. g. sheet-iron were bolted. At the upper end a 12 by 12 by 24-in., wooden block was fastened vertically, and the whole was weighted so that it was nearly submerged. On the upper end of the float there were iron straps which carried two heavy hooks for fastening the lifting tackle, and a light rod on which could be put a flag during the day or a lantern at night. The second type of float gave the greater satisfaction, as it was the more substantial for this heavy service, and its motion was not affected appreciably by the wind.

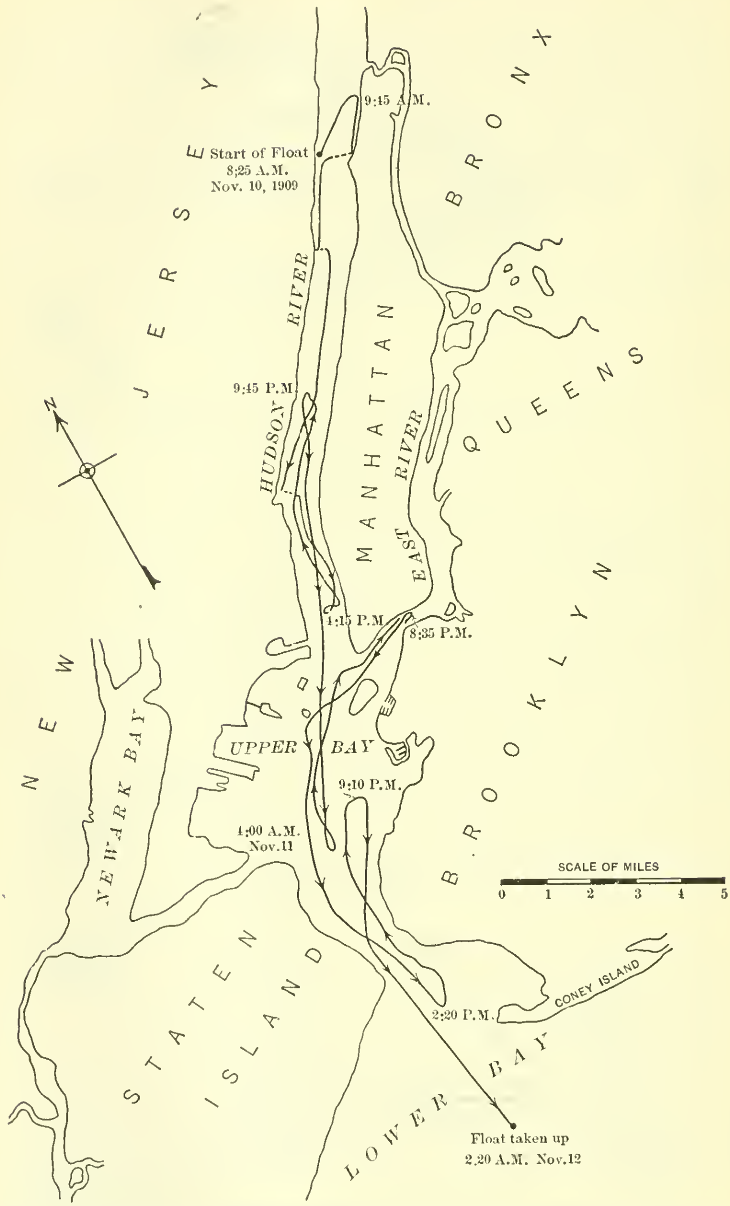
Floats were set adrift in different parts of the harbor and at different stages of the tide. They were followed by observers who noted their position at frequent intervals, generally using the sextant, and the positions were plotted on a large-scale chart. Many of the paths thus traced were duplicated by other floats. Broadly speaking, the floats showed a general trend toward the sea, past Sandy Hook; although, naturally, they were carried forward and backward with the ebb and flood tides. Figs. 21 to 32, inclusive, show the paths of twelve representative floats, as observed by the Metropolitan Sewerage Commission. Fig. 33 shows the paths of floats at the entrance to Jamaica Bay as observed under the direction of Col. John G. D. Knight, Corps of Engineers, U. S. Army, in December, 1908. Some were started in the ocean and entered the bay on flood tide, others were set adrift in the entrance and passed out on ebb tide. The tendency is for the currents to flow into and out of the bay around Rockaway Point. The flow is chiefly to and from the ocean, and less directly past Coney Island.

The paths of the floats show the general behavior of the currents in the harbor; the general drift toward the Atlantic Ocean; the backward and forward paths in the Upper Bay and rivers; the tendency to hug the Staten Island shore at The Narrows; the tendency of a particle to remain in Newark Bay and Kill van Kull; and the tendency of a particle to remain in the East River (Figs. 27, 28, 29, and 30); which substantiates the conclusion that the southerly resultant flow of the East River is not great.

When velocity data are worked out from float experiments, a correction should be made for the range of tide existing on the day of

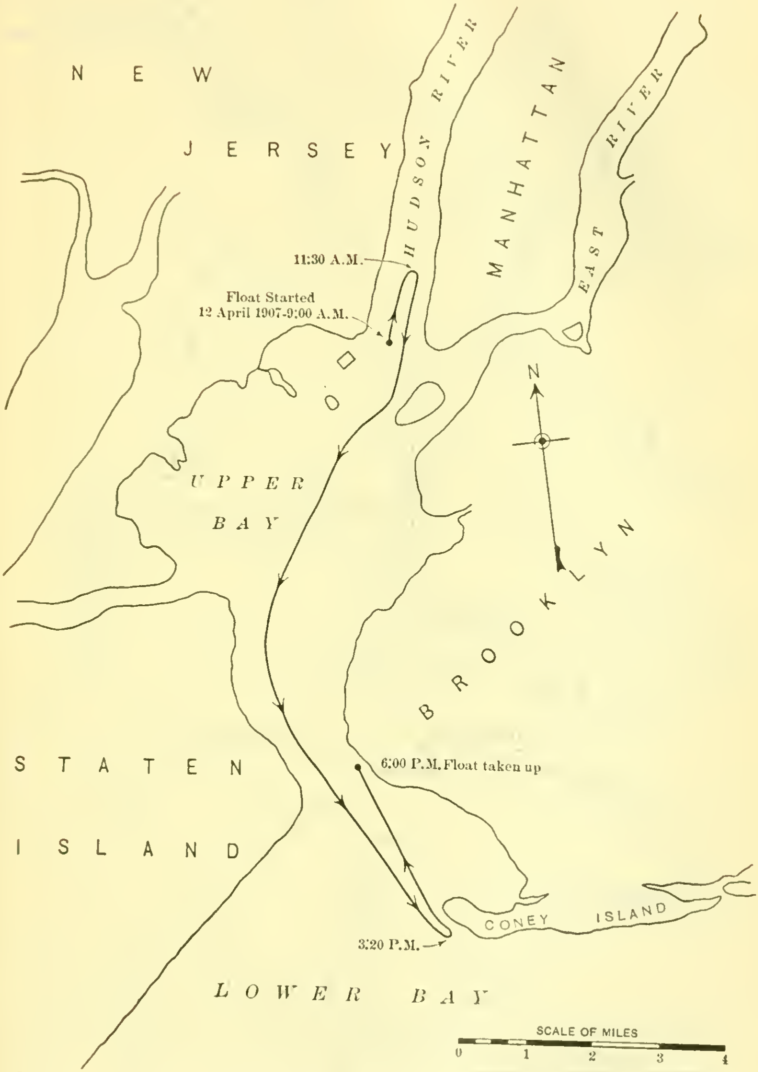
PATH OF A FLOAT FROM NEWARK BAY TO THE SEA.

FIG. 21.



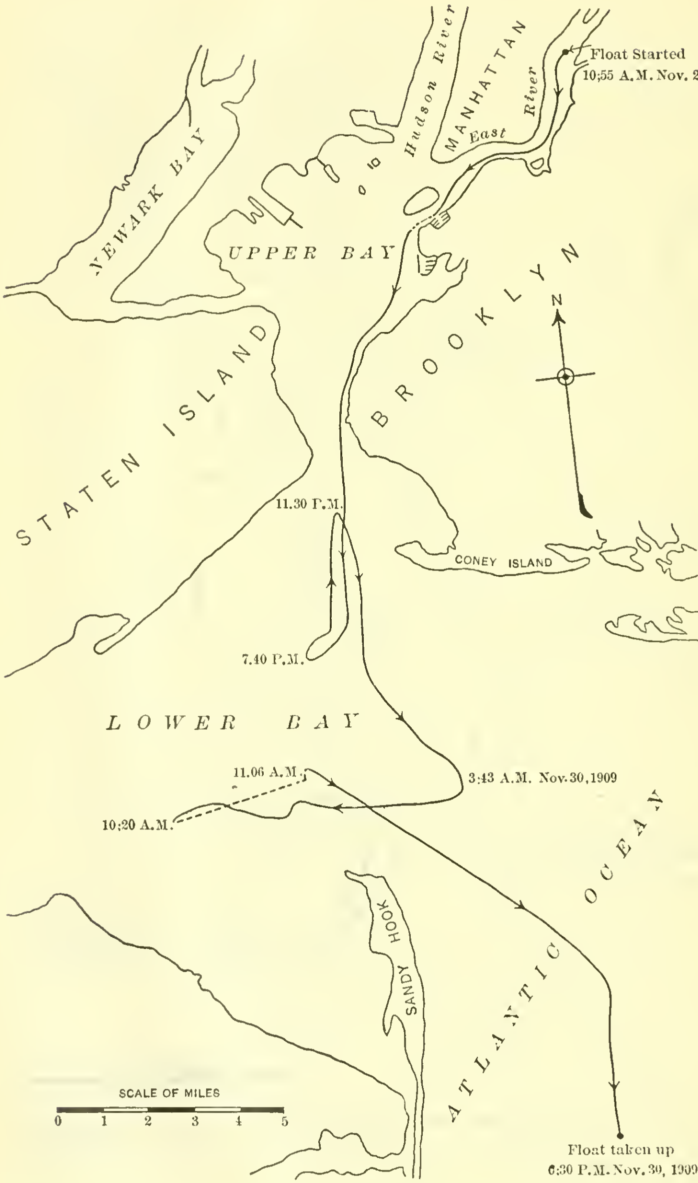
PATH OF A FLOAT FROM THE HUDSON RIVER TO THE LOWER BAY

FIG. 22.



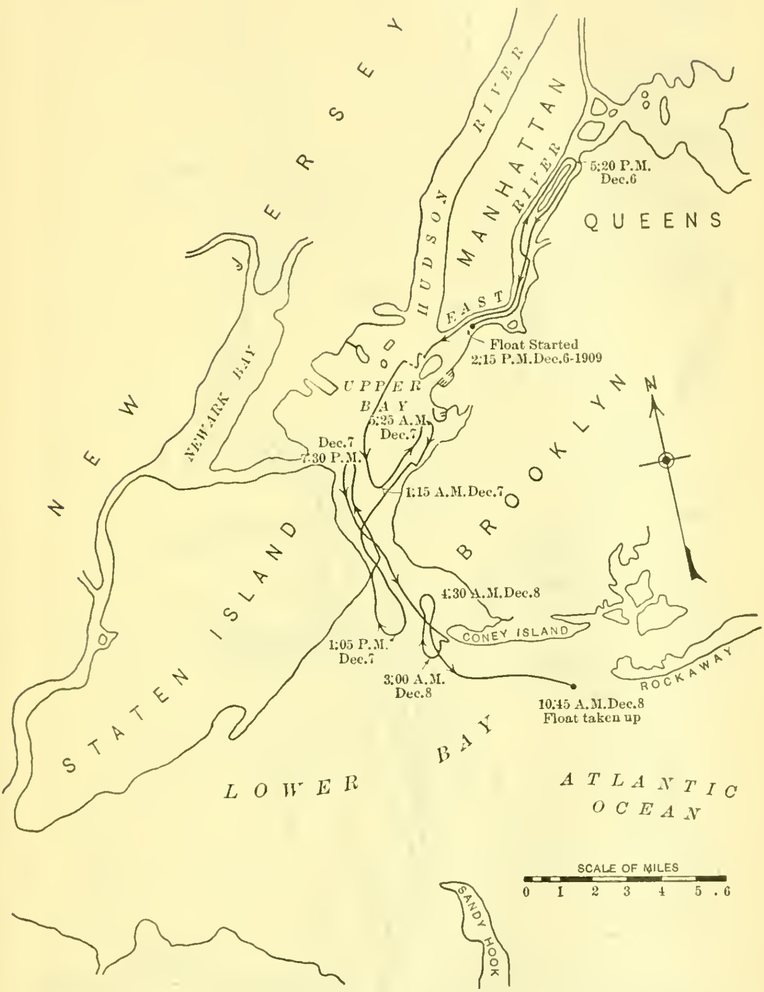
PATH OF A FLOAT FROM THE HUDSON RIVER TO THE LOWER BAY

FIG. 23.



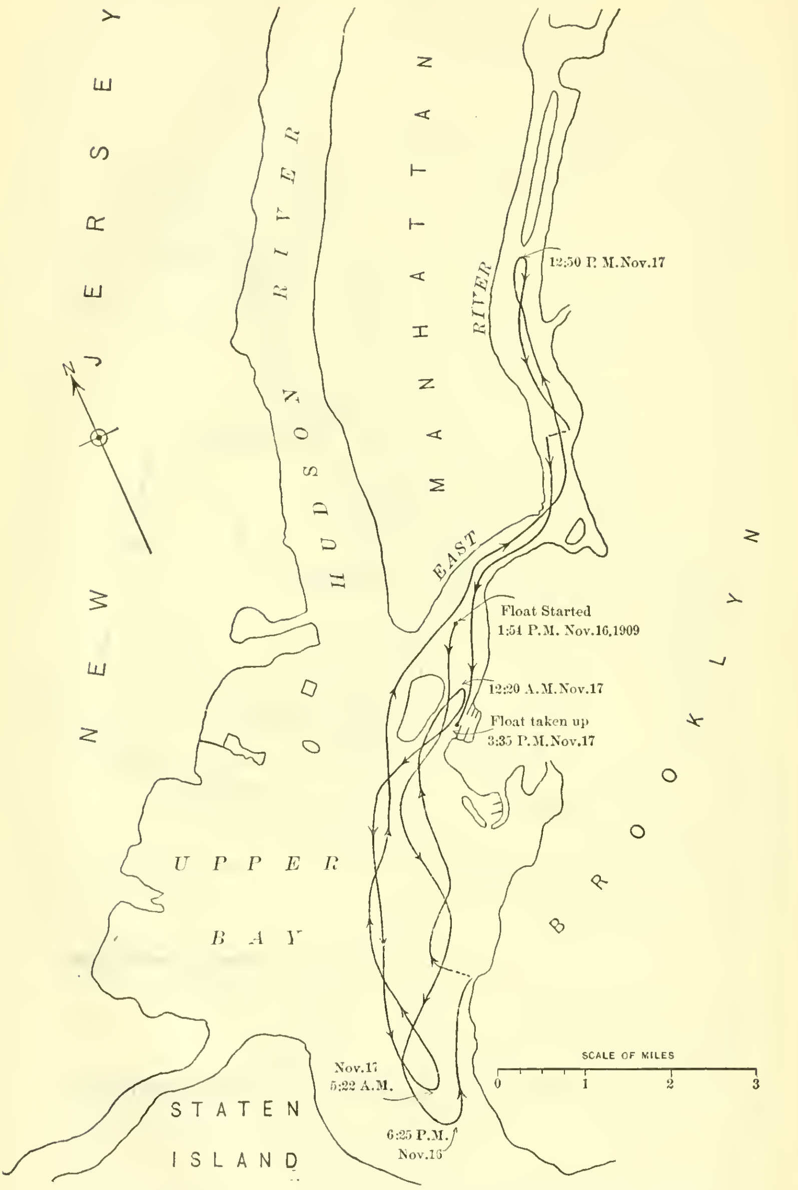
PATH OF A FLOAT FROM THE EAST RIVER TO THE SEA

FIG. 24.



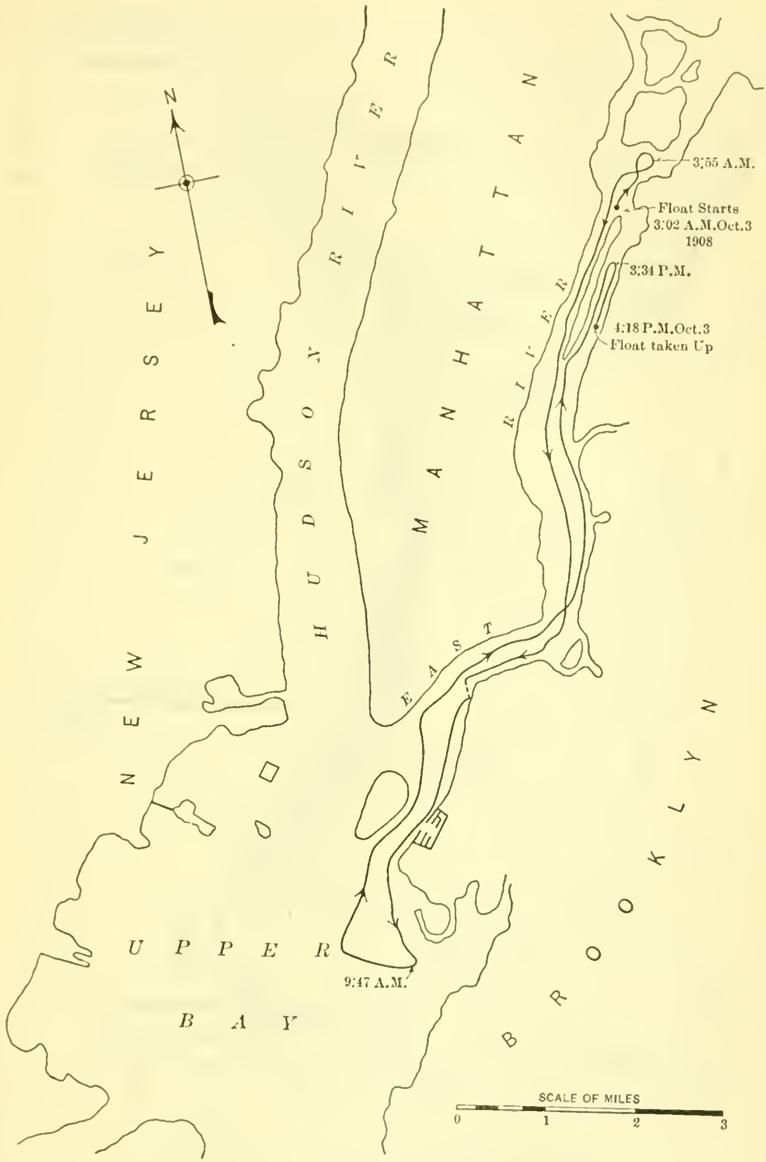
PATH OF A FLOAT FROM THE EAST RIVER TO THE LOWER BAY

FIG. 25.



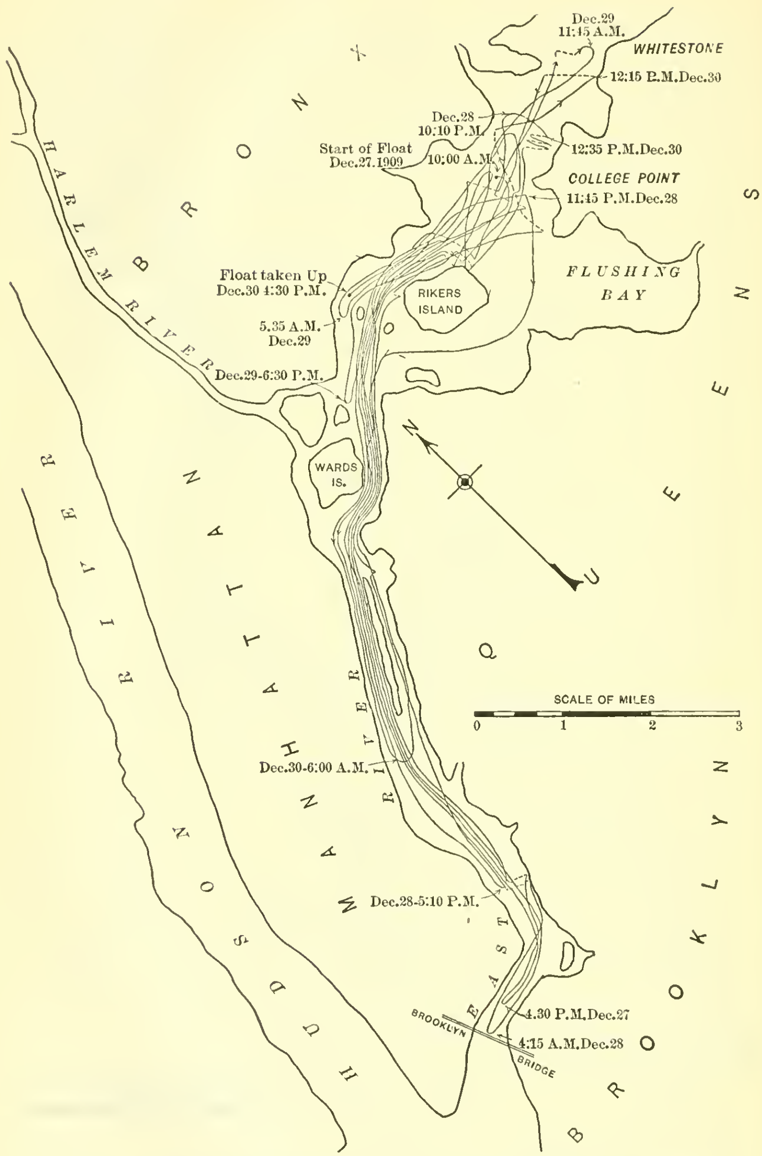
PATH OF A FLOAT IN THE UPPER BAY AND EAST RIVER

FIG. 26.



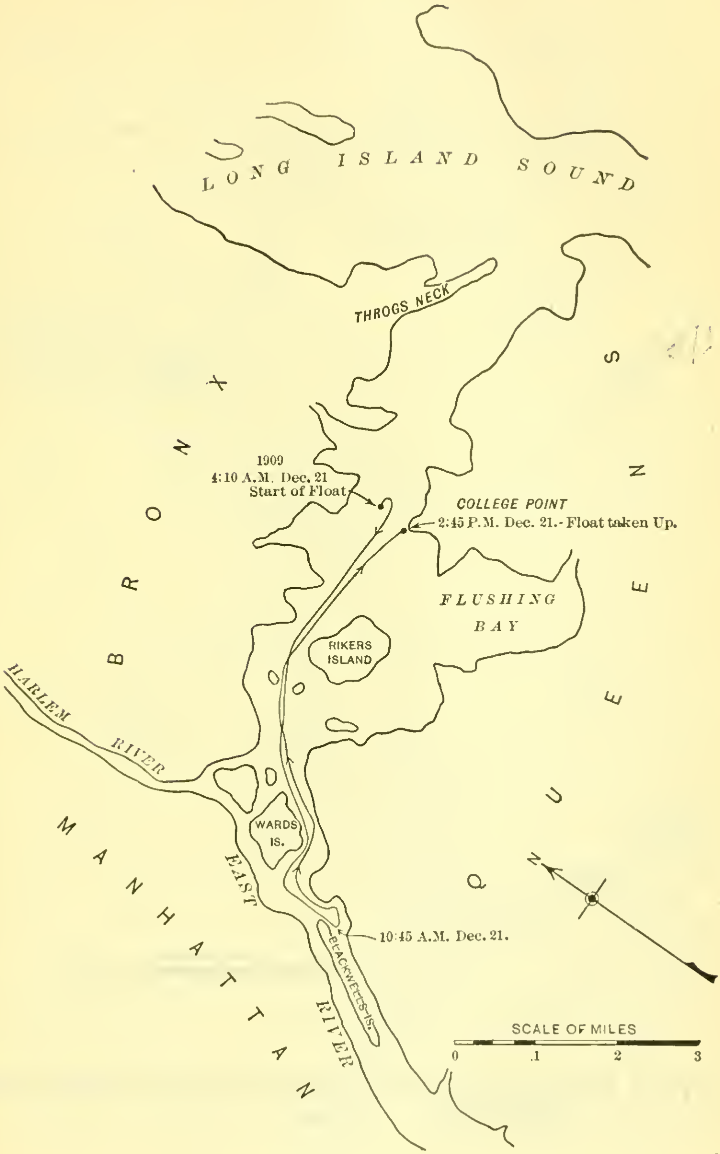
PATH OF A FLOAT IN THE UPPER BAY TO THE EAST RIVER

FIG. 27.



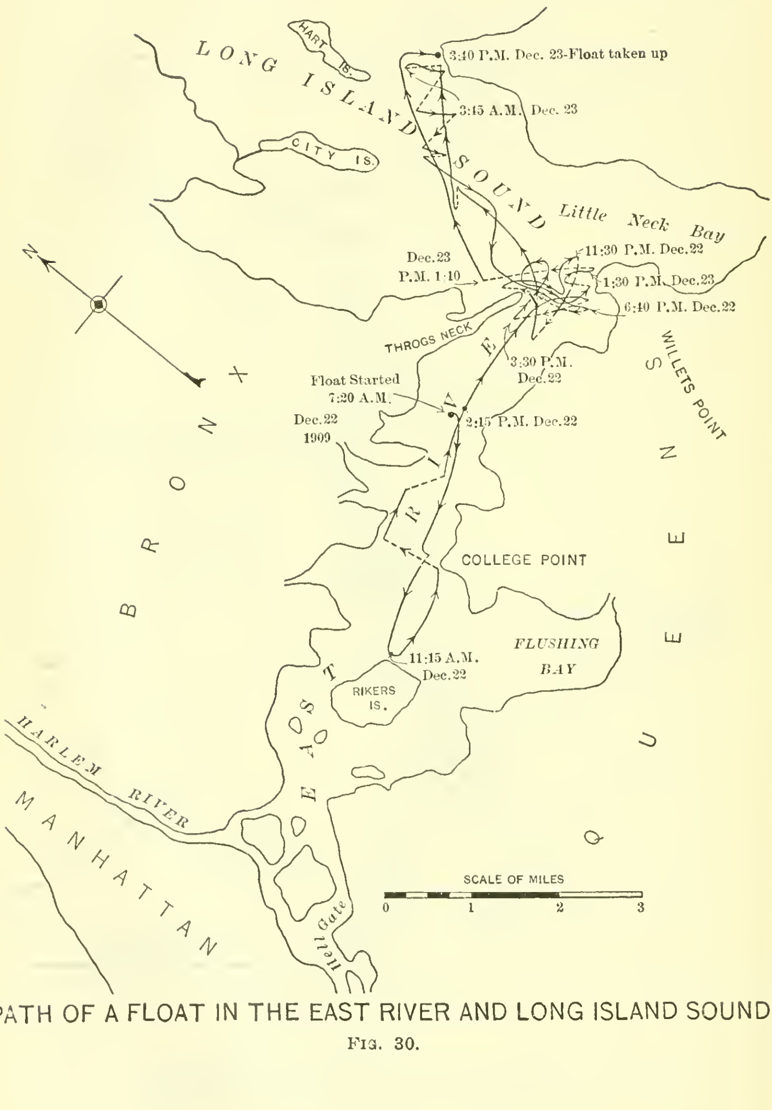
PATH OF A FLOAT IN THE EAST RIVER

Fig. 28.



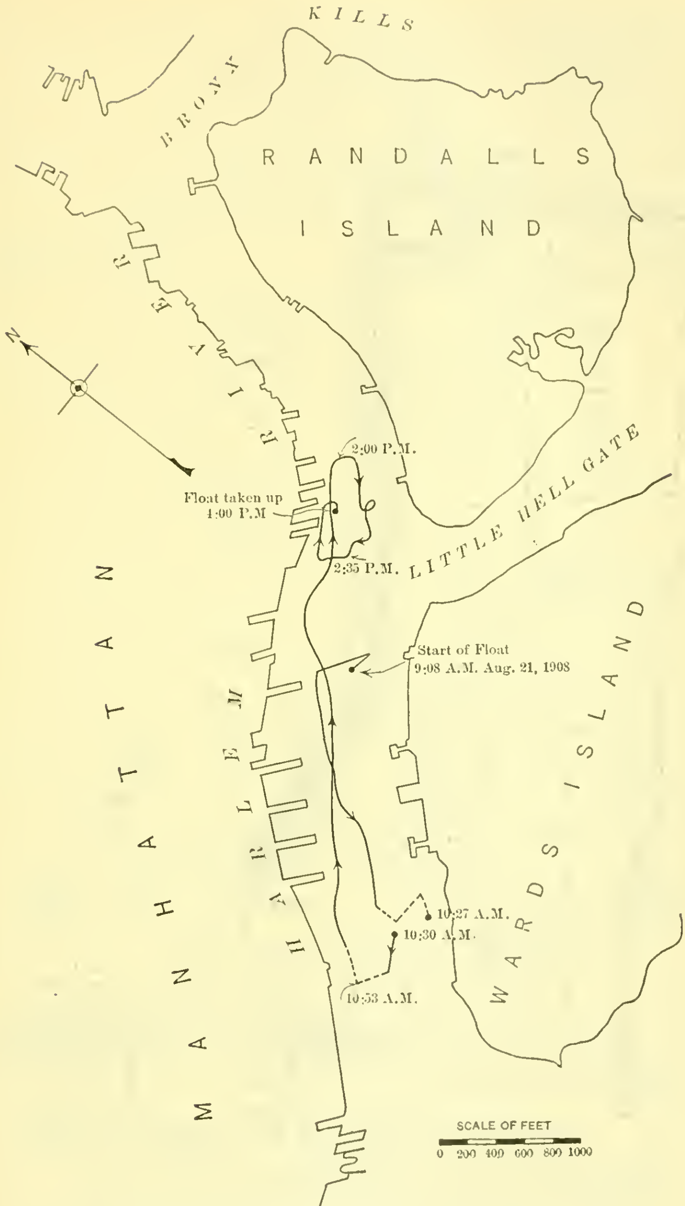
PATH OF A FLOAT IN THE EAST RIVER.

FIG. 29.



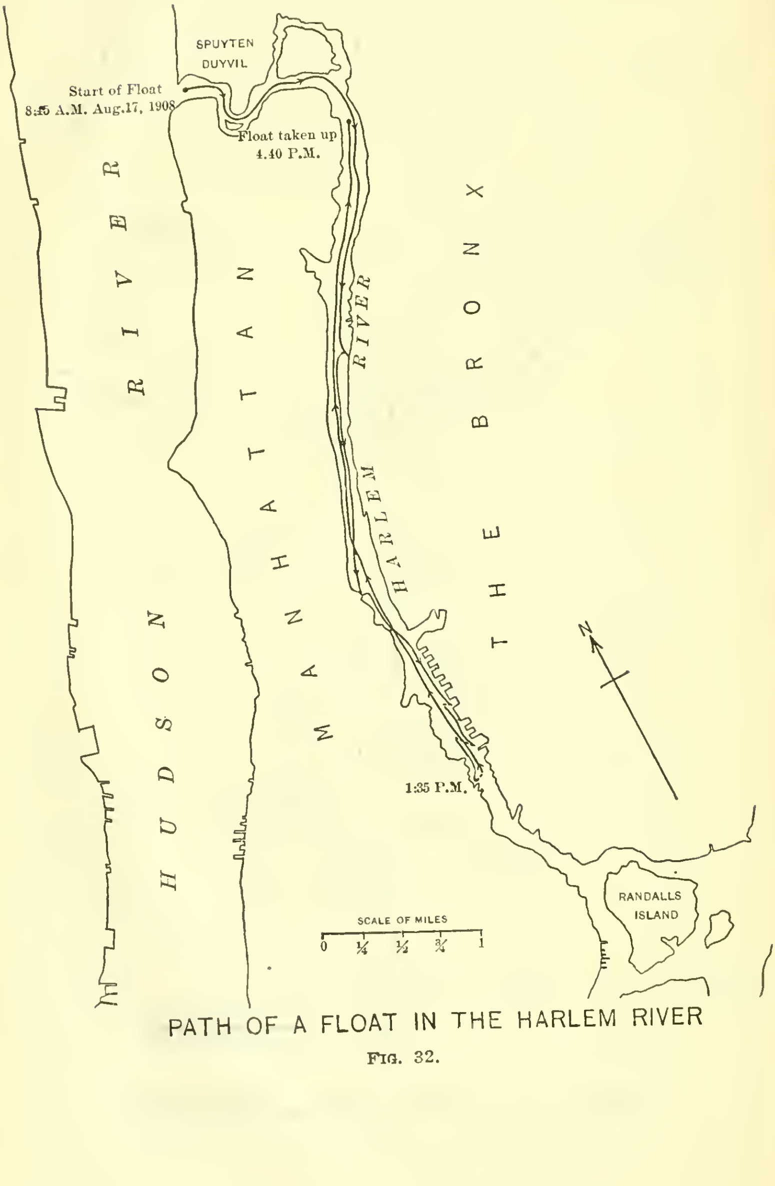
PATH OF A FLOAT IN THE EAST RIVER AND LONG ISLAND SOUND

Fig. 30.



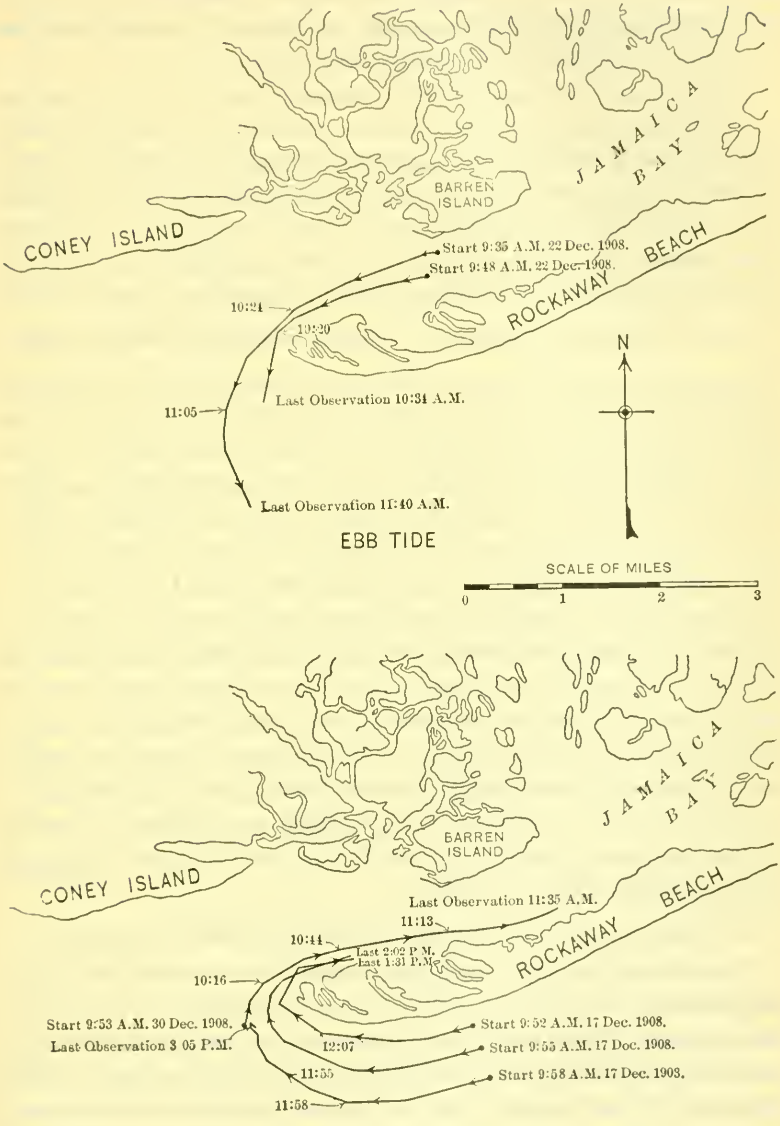
PATH OF A FLOAT IN THE HARLEM RIVER

FIG. 31.



PATH OF A FLOAT IN THE HARLEM RIVER

FIG. 32.



FLOOD TIDE

PATHS OF FLOATS THROUGH ROCKAWAY INLET.

AS OBSERVED UNDER THE DIRECTION OF COL. JOHN G.D. KNIGHT, CORPS OF ENGINEERS, U.S.A.

FIG. 33.

observation, in order to change the observed velocity to that which would have existed for a normal range, as discussed under the heading, "Effect of Tidal Range on Velocity," page 670.

Calculations were made of the theoretical paths which particles would take during successive tides, as explained on page 662. Such paths are shown in Fig. 34 for a particle starting in the Hudson River, off Grant's Tomb, on a flood tide; and in Fig. 35, starting off College Point, in the East River, one on flood and one on ebb tide. The paths were worked out by calculating the distance traveled by a particle during a complete flood or ebb period by substituting in the formula, $\frac{2}{\pi} A\ t$, in which A denotes the velocity, in knots at strength, and t the duration of the current, in solar hours, both as given in the Tide Tables. The distances traveled on both ebb and flood currents were thus determined for each mile, and the results arranged in proper order. The plottings of the paths in Figs. 34 and 35 were then made by using the average of the results of the calculated distances for the miles which the particle would pass during the current being considered. The path calculated for the Hudson River and Upper Bay, Fig. 34, agrees closely with that actually traversed by a float, as shown in Fig. 22. The paths calculated for the East River were longer than the distances traversed by the floats on each current, which was to be expected, as the calculation is based on the particle remaining in the swiftest thread. However, Fig. 35 indicates that a particle set adrift in the East River near College Point is likely to remain in the river, as was the result with a float, Fig. 28, which was followed for nearly $3\frac{1}{2}$ days and never passed farther south than the Brooklyn Bridge, nor farther east than Whitestone Point.

Volumes of Water Flowing on Flood and Ebb Currents.

The volume flowing on each tidal current can be estimated in accordance with the principles deduced for calculating the flow of rivers. The estimates will be approximately correct, and considering the daily variations, will be sufficiently accurate for all practical purposes. These estimates are based on the area of a selected section below the mean level of the water surface, the mean velocity of the current, and the time during which the current flows.

It is neither a simple nor an easy matter to make gaugings of the volumes flowing on the tidal currents, because the volumes are so large,

and the measurements would have to extend over long periods, in order to average out the daily variations; and they would have to be made at stations located so closely together as to interfere with navigation. For exact results, current velocity measurements would have to be taken at many different depths at each station. Such measurements should be made nearly coincident at each station, and repeated at short intervals of time, because the velocities of the threads at different depths may vary, and are subject to changes in direction which are not synchronous, due to the reversals of the tidal currents. The work of making actual gauge measurements of such large volumes as flow in the East River, the Hudson River, and through The Narrows would be very costly.

The volumes of the currents passing any selected section during a tidal interval are not constant, because of lunar and seasonal variations. The average or mean volumes, however, can be estimated with sufficient accuracy for all practical purposes.

With the exception of the East River and the Harlem River, where the currents are purely hydraulic, that is, produced by a difference in level of the water surfaces at each end, the main currents are complicated by the discharges of the land-waters from the water-sheds above. In consequence, the ebb currents exceed the flood currents in volume by just the discharges of land-water. This is true under mean conditions, that is, when the ranges of consecutive tides are the same.

Therefore, each tidal flow is composed of a periodic portion due to the true tidal wave, and a non-periodic portion due to the land-water discharge. These two portions are mixed into one volume, but, for purposes of estimate, must be kept distinct. It will be clear, then, that the resultant discharge from the harbor to the sea will be the land-water discharges of the rivers, and no more, except the southerly net flow of the East River, which, as will be shown later, is not great.

The rise and fall of the tides and the current velocities may be assumed to vary according to the ordinates of a sinusoid. Any deviation is due to local conditions or to temporary irregularities. The true tidal actions are periodic, and each tide extends over a period of 6 lunar hours or (nearly) 6.21 solar hours.

The U. S. Coast and Geodetic Survey has made estimates of the volumes of the tides flowing past selected sections, for each month, by

SURFACE CURRENT MOVEMENTS DURING SUCCESSIVE TIDES THROUGH THE HUDSON RIVER, UPPER AND LOWER NEW YORK BAYS.

CALCULATED FROM VELOCITIES GIVEN IN TIDE TABLES FOR 1908.

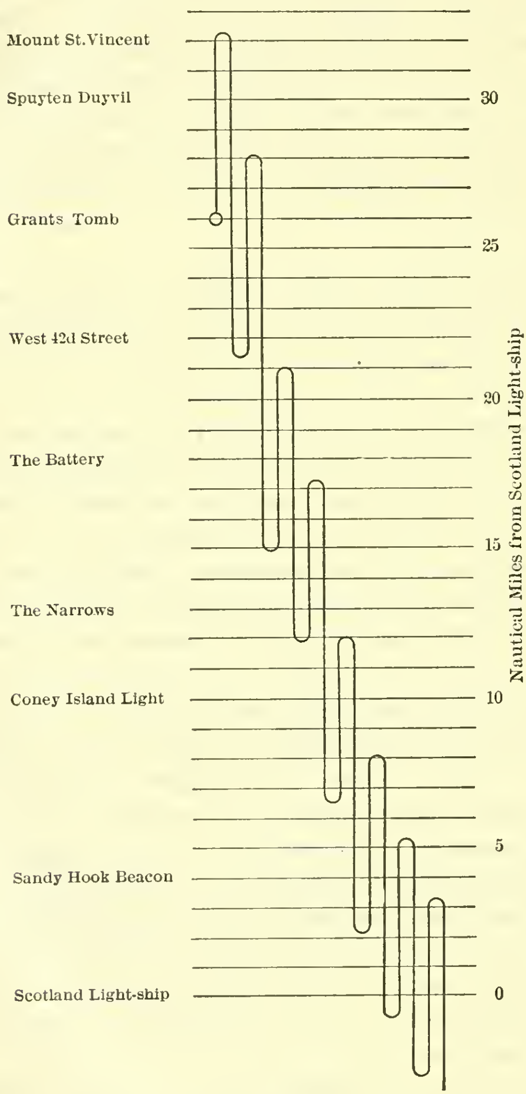


FIG. 34.

SURFACE CURRENT MOVEMENTS DURING SUCCESSIVE TIDES THROUGH THE EAST RIVER, UPPER AND LOWER NEW YORK BAYS.

CALCULATED FROM VELOCITIES GIVEN IN TIDE TABLES FOR 1908.

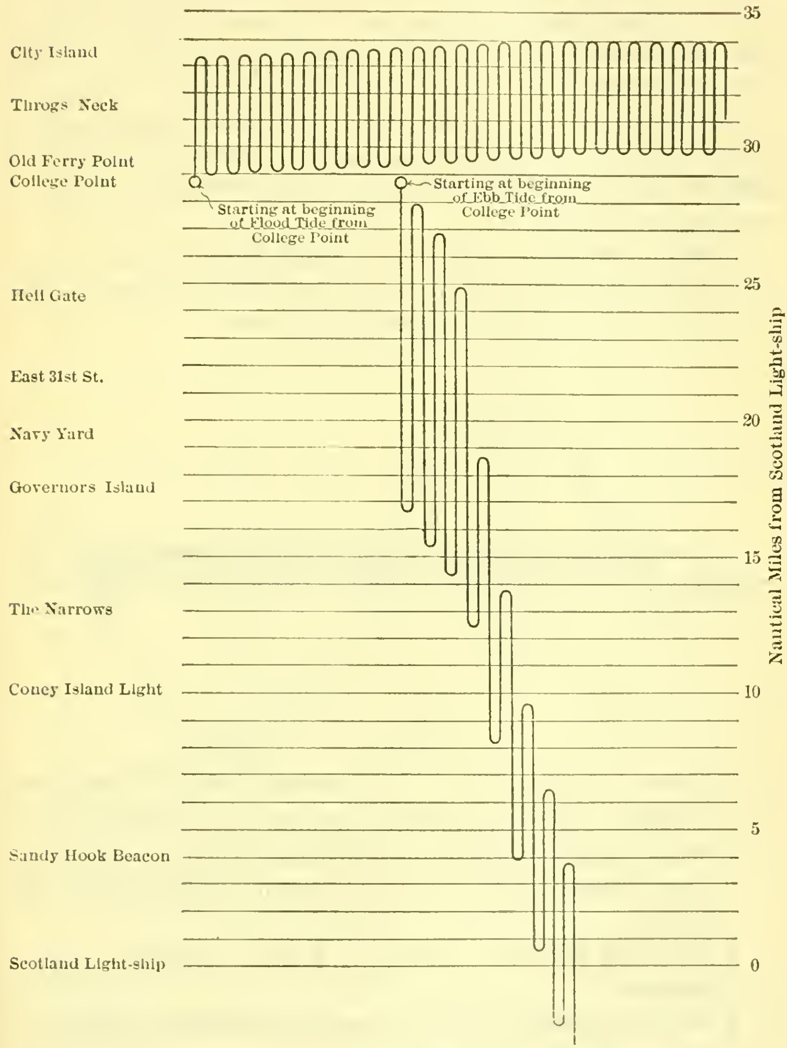


FIG. 35.

using the monthly mean discharge of land-water, and then averaging these monthly results for the yearly mean.

Draw a line, *A B C*, Fig. 36, and divide it into lunar hours. Draw a curve of sines, *A M B C*, such that *M N* represents the maximum rate of flow due to the periodic portion. Its value is found by multiplying the area of the section below mean water level by the mean sectional velocity of the current at the time of strength. The result is usually given in cubic feet per second. The mean sectional velocity is about 0.75 times the velocity at the surface. Distances along *A B C* represent lunar hours and also degrees of angles. Thus *A B* is 6 lunar hours and 180 degrees.

Parallel to, and above, *A B C*, draw a line, *D E F*, so that the distance between the two will represent the rate of flow of the non-periodic portion, or the river discharges of land-water.

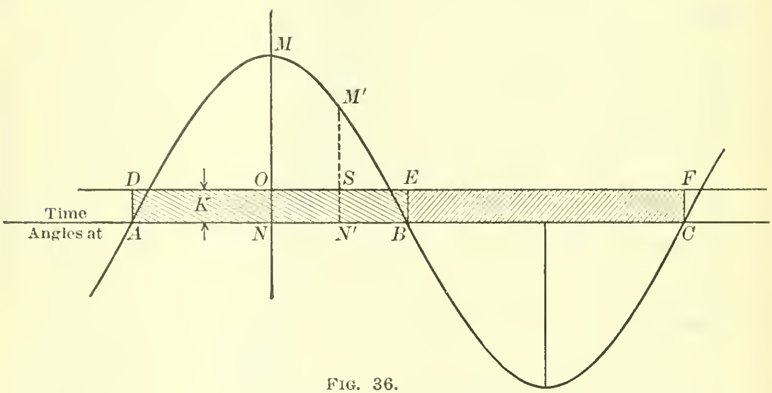


Fig. 36.

Let R denote the rate of flow of the periodic portion at maximum strength, or $M\ N$, in cubic feet per second (or per solar hour);

" R' denote the rate of flow of the periodic portion at any time, $N\ N'$, after strength, or $M'\ N'$, in cubic feet per second (or per solar hour);

" t denote the time after strength, or $N\ N'$, in seconds (or in solar hours);

" a denote a constant, so that $a\ t$ equals the number of degrees in the angle between N and N';

" K denote the rate of flow of the non-periodic portion, or $O\ N$, in cubic feet per second (or per solar hour);

" θ denote the number of degrees in the angle between O and E, or where R' equals K.

Then:

$$M'\ N' = R' = R \cos.\ a\ t$$

$$M'\ S\ = y = R \cos.\ a\ t - K.$$

As θ is the angle at which $R \cos.\ a\ t = K$, $\cos.\ \theta = \dfrac{K}{R}$

$$O\ E = \text{half duration of flood} = \frac{\theta}{a}.$$

Therefore,

$$\text{the area, } O\ M\ M'\ E = \int_0^\theta y\ dt = \int_0^\theta (R \cos.\ a\ t - K)\ dt$$

$$\text{or, the volume of half flood} = \int_0^\theta R \cos.\ a\ t\ dt - K\ dt$$

$$= \left[\frac{R}{a} \sin.\ a\ t - K t \right]_0^\theta$$

$$= \frac{R}{a} \sin.\ \theta - K\ \frac{\theta}{a} - \frac{R}{a} \sin.\ O + K\ \frac{O}{a} \ldots\ldots\ldots \ (1)$$

As $\sin.\ \theta = \sqrt{1 - \cos.^2\ \theta}$, or $\sqrt{1 - \dfrac{K^2}{R^2}}$, and as θ is the angle the cosine of which is $\dfrac{K}{R}$, Equation (1) becomes

$$= \frac{R}{a} \sqrt{1 - \frac{K^2}{R^2}} - K\ \frac{\cos.^{-1} \dfrac{K}{R}}{a}$$

$$= \frac{1}{a} \left[R \sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-1} \frac{K}{R} \right] \ldots\ldots\ldots\ldots (2)$$

For time in seconds,

$$a = \frac{\pi}{\text{number of seconds in 6 lunar hours}} = \frac{\pi}{22\ 357}.$$

or, for solar hours,

$$a = \frac{\pi}{\text{number of solar hours in 6 lunar hours}} = \frac{\pi}{6.21},$$

therefore, the volume flowing on the total flood is twice Equation (2).

After substituting the value of a, for the time in seconds, the total flood volume

$$= 14\ 233 \left[R \sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-1} \frac{K}{R} \right] \ldots\ldots\ldots\ldots (3)$$

The volume flowing on the total ebb is the flood volume plus the run-off for 12 lunar hours. This latter quantity is represented by the area, $A\ B\ C\ F\ E\ D$, or by $\frac{2\ \pi\ K}{a}$, as $A\ C = \frac{2\ \pi}{a}$.

The volume flowing on the ebb, therefore, is,

$$= \frac{2\ K\ \pi}{a} + 14\ 233 \left[R \sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-1} \frac{K}{R} \right]$$

$$= 44\ 714\ K + 14\ 233 \left[R \sqrt{1 - \frac{K^2}{R^2}} - K \cos.^{-1} \frac{K}{R} \right] \ldots\ldots (4)$$

Taking a section between Sandy Hook and Coney Island, the value of R is 1 685 812 cu. ft. per sec.,* being the product of the mean cross-sectional area and 0.75 of the average surface velocity. The value of K, or the rate of run-off, is variable for each month, but the monthly means are given in Table 5. Substituting these values in Equations (3) and (4), and working out the volumes of flood and ebb for each month, the mean flood and ebb volumes can be estimated by averaging the monthly results.

The mean total ebb volume is 24 658 499 000 cu. ft.
The mean total flood volume is 23 322 888 000 " "

A section was taken across The Narrows, and Equations (3) and (4) were solved for each monthly result; the details are given in Table 6. In this way the flood and ebb volumes for each month can be ascertained. The mean result for the year has been calculated, and the averages for the 12 months are also given.

The velocities of current at strength have been taken from Table 4. The values of K have been found by adding the resultant flow of the East River, which is about 40 000 000 cu. ft. per 6 lunar hours (page 715), to the total of the land-water entering the Upper Bay above The Narrows, as given in Table 5, and dividing the sum by 22 357, or the number of seconds in 6 lunar hours.

The mean volumes of the normal tidal flows past any section can be estimated closely by a shorter process: Find the mean height of the curve, AMB, Fig. 36, for both flood and ebb currents; then multiply this average rate of flow by the area of the section and by the duration of the current in time, and the result will be the mean of the volumes

* From information received from the U. S. Coast and Geodetic Survey, January 22d, 1910.

TABLE 6.—SECTION AT THE NARROWS.

Data: Width = 5 280 ft.
Average depth = 61 ft.
Area of section = 322 000 sq. ft.
Velocity at strength, flood = 1.8 knots.
" " " ebb = 2.1 "
" " " mean = 1.95 " = 3.3 ft. per sec.

R = area × 0.75 × velocity at strength (mean of ebb and flood).

R = 800 000 cu. ft. per sec.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Month.	K	$R\sqrt{1-\frac{K^2}{R^2}}$	$\text{Cos.}^{-1}\frac{K}{R}$	$K\cos.^{-1}\frac{K}{R}$	Column 3 minus Column 5.	Column 6 × 14 233. Flood volume.	Column 2 × 44 714.	Column 7 plus Column 8. Ebb volume.
Jan.	21 510	799 712	1.5442	33 216	766 496	10 950 000 000	961 798 000	11 911 798 000
Feb.	17 240	799 824	1.5493	26 710	773 114	11 004 000 000	770 860 000	11 774 860 000
Mch.	57 700	797 912	1.4988	86 481	711 431	10 127 000 000	2 579 998 000	12 706 998 000
Apr.	59 720	797 768	1.4960	89 341	708 427	10 084 000 000	2 670 200 000	12 754 200 000
May.	31 740	799 368	1.5313	48 603	750 765	10 686 000 000	1 419 150 000	12 105 150 000
June.	26 110	799 576	1.5382	40 162	759 414	10 810 000 000	1 167 350 000	11 977 350 000
July.	17 390	799 808	1.5491	26 939	772 869	11 001 000 000	777 530 000	11 778 530 000
Aug.	15 720	799 848	1.5511	24 383	775 465	11 038 000 000	702 870 000	11 740 870 000
Sept.	17 630	799 808	1.5488	27 305	772 503	10 996 000 000	788 270 000	11 784 270 000
Oct.	23 830	799 648	1.5412	36 727	752 921	10 718 000 000	1 065 400 000	11 783 400 000
Nov.	22 180	799 696	1.5431	34 226	755 370	10 752 000 000	991 680 000	11 743 680 000
Dec.	28 010	799 504	1.5360	43 023	756 481	10 768 000 000	1 252 350 000	12 020 350 000
Mean...	28 230	799 496	1.5358	43 356	756 140	10 762 000 000	1 262 200 000	12 024 200 000
Average for 12 months						10 744 500 000	.....	12 006 800 000

flowing through the section on both ebb and flood currents. The flood volume will be the mean, thus found, less one-half of the land-water discharged during 12 lunar hours; and the ebb volume will be the mean volume plus one-half of the land-water.

The writer thus estimated the volumes flowing on the tidal currents past various sections of the harbor, chiefly based on investigations made by the Metropolitan Sewerage Commission. These results agree fairly well with the estimates made by the United States Coast and Geodetic Survey.

For comparison, the tidal volumes at The Narrows are given in cubic feet:

	Ebb.	Flood.
By short method..........	12 041 200 000	10 778 800 000
" Equations (3) and (4).	12 024 200 000	10 762 000 000
" U. S. C. and G. Survey.	12 211 187 000	11 028 853 000

The U. S. Coast and Geodetic Survey results were calculated by Equations (3) and (4), but were based on a mean velocity at strength of 2 knots, with no allowance for any resultant discharge from the East River.

Volumes of Tidal Discharges.

The Narrows.

The section between Fort Wadsworth and Fort Hamilton is 5 280 ft. wide, and its average depth below mean water level is 61 ft., so that its area is 322 000 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.34 knots, and on flood 1.17 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission and drawing a smooth curve through the points. The areas below these approximate sinusoids were then divided by their lengths, giving the average velocities. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.255 knots, or 7 631 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75. The mean cross-sectional velocity, therefore, is 5 723 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 1 837 000 000 cu. ft. per solar hour as the mean rate of flow through the section.

The length of time that each current flows was taken at 6 lunar hours, or 6.21 solar hours. The mean volume flowing on ebb and flood currents is 11 410 000 000 cu. ft., obtained by multiplying the mean rate of flow through the section by the length of time.

The mean yearly discharge of land-water run-off past The Narrows for 6 lunar hours is 591 200 000 cu. ft., or 1 182 400 000 cu. ft. per tidal cycle of 12 lunar hours. These figures are from the records of the U. S. Coast and Geodetic Survey and the New York State Engineer and Surveyor. See Table 5.

In addition to this run-off, there is a net discharge through the East River from the Sound into the Upper Bay which passes out through The Narrows. This net discharge is about 80 000 000 cu. ft. per tidal cycle, as explained on page 715.

The total net discharge, therefore, is 1 262 400 000 cu. ft.; and, from what has been stated on page 708, the average ebb volume will be approximately the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 12 041 200 000 cu. ft.
Average volume of flood current, 10 778 800 000 " "

Hudson River.

The section of the river opposite West 39th Street, Manhattan, is 3 900 ft. wide, and its average depth below mean water level is 37.44 ft., so that its area is 146 000 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.86 knots, and on flood 1.26 knots. These velocities were obtained by plotting curves from information obtained from the U. S. Coast and Geodetic Survey and from the U. S. Coast Pilot, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.56 knots, or 9 485 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75. The mean cross-sectional velocity, therefore, is 7 113 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 1 038 000 000 cu. ft. per solar hour as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 6 447 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

The mean yearly discharge of land-water run-off past Battery Place for 6 lunar hours is 543 600 000 cu. ft., or 1 087 200 cu. ft. per tidal cycle of 12 lunar hours. These figures are from the records of the U. S. Coast and Geodetic Survey and the New York State Engineer and Surveyor. See Table 5.

From what has been stated on page 708, the average ebb flow will be the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 6 990 600 000 cu. ft.
Average volume of flood current, 5 903 400 000 " "

East River.

The run-off from the areas which naturally drain into the East River is small compared with the volumes of the tidal discharges. As the East River is a strait, the flow is subject to the actions of the separate tidal waves entering at each end, Throgs Neck and Governors Island.

The difference in water level creates a hydraulic flow through the strait or river. As the "westerly and southerly" and the "northerly and easterly" hydraulic slopes are equal, there would be no reason to expect a greater flow in one direction than in the other.

A sufficient number of actual measurements of velocity, and extending over sufficiently long periods of time, have not been made to determine positively the mean volumes flowing on the two tidal currents; therefore, the difference between the volumes can only be estimated by theoretical reasoning.

If a base line is drawn and divided into equal spaces, each of which represents a lunar hour, and if a sinusoid be drawn on this base line to represent the rise and fall of the water above and below mean water level off Willets Point, then each ordinate would represent the surface level of the water above or below mean water level at each lunar hour.

If a second sinusoid be drawn in a similar manner for the rise and fall of the tide at Governors Island, then, by placing it so as to be spaced at the proper interval of time relative to the Willets Point

sinusoid, a comparison can be made of the elevations of the water surface at both ends of the strait for every lunar hour. These curves are represented in Fig. 7, and from a study of them it will be noted that, from the lunar hours of IX to III after the transit of the moon, the surface of the water at Willets Point is above that at Governors Island, and a reverse condition is maintained from the lunar hours of III to IX. These two sinusoids form lunes, the ordinates of which represent the difference of the water levels between the ends of the straits at each lunar hour. If the centers of these lunes are determined, it will be noted that, during the time when the current is flowing from Willets Point to Governors Island, the water level is 0.74 ft. above mean water level. For the reverse current, from Governors Island to Willets Point, the average water level is the same distance below mean water level. In other words, the cross-sectional area of the stream during the "westerly and southerly" flow would be greater than during the "northerly and easterly" flow; or, a greater flow would occur from the Sound to the Upper Bay than in the reverse direction, if the velocities were the same in each direction.

Assuming that the average depth of the river below mean water level is 38 ft., then the average depth during the southerly flow would be

$$38 + 0.74 = 38.74 \text{ ft.},$$

and in a similar way the average depth during the northerly flow would be

$$38 - 0.74 = 37.26 \text{ ft.}$$

If the velocities are the same in each direction, and the lengths of time during which the currents flow are equal, then the mean volumes flowing on a southerly current would be

$$\frac{38.74}{37.26} = 1.04$$

times the volume flowing on a northerly flow. In other words, the southerly discharge of the river would be, from this reasoning, about 4% greater than the northerly discharge.

If, at different sections of the river between Willets Point and the Battery, sinusoids are drawn to represent the surface of the water above and below mean water level, and on these diagrams the northerly and southerly currents are marked, and the average water levels are determined for each current, it will be found that the cross-sectional

area of the river is greater on the southerly current for those sections between Willets Point and some place at about 60th Street; but for the sections between the Battery and about 60th Street, the reverse condition obtains. From this it would seem as if the southerly flow did not exceed the northerly flow by the quantity which would be expected from the difference in water level at each end of the strait.

Observations made by the U. S. Coast and Geodetic Survey indicate that, generally speaking, the current having the larger section has the smaller velocity, and *vice versa*, thus leaving the question of the quantity of net discharge uncertain.

Henry Mitchell* gives, as the result of some of his observations, the volumes passing through a section opposite Wall Street, 4 341 100 000 cu. ft. on flood and 4 383 500 000 cu. ft. on ebb, or a difference of only 1% in favor of the southerly flow.

The Coast Survey's observations for sections between Wallabout Bay and the southern extremity of Blackwells Island indicate† that the flood volume is quite as great as that of the ebb. These observations covered work done at 18 stations. The Survey's observations between Blackwells Island Light and Throgs Neck, taken at 21 stations, indicate† that the easterly current is decidedly stronger than the westerly, but that the average cross-sectional area of the currents is greater on the westerly flow. Thus the westerly net discharge cannot be great, for these figures would indicate a small net discharge easterly into the Sound.

The Survey's observations around Blackwells Island, taken alone, indicate‡ a small net southerly discharge, the west-and-south-going volume being about 3% greater than the north-and-east-going volume. As Blackwells Island is about midway between the ends of the strait, and as the stream is well defined at this point, these observations seem to be important in deciding the question of whether there is or is not a net westerly and southerly discharge.

Although Henry Mitchell lays great stress on the difference in cross-sectional areas, which at Hell Gate amounts to nearly 8%, and thus reasons that the westerly discharge is about that much greater than the easterly discharge, the theoretical state of the conditions indicates

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* Report of the U. S. Coast and Geodetic Survey, 1871, p. 131.

† U. S. Coast and Geodetic Survey, Letter of February 6th, 1909.

‡ U. S. Coast and Geodetic Survey, February 6th, 1909.

that the volume flowing on the currents will not vary directly as the cross-sections of the two streams at Hell Gate. This is true, because, the smaller the section at the Gate, the greater must be the velocity at that point, as the length through the Gate is a small fraction of the lengths of the other reaches of the river. This result can be shown by Bernoulli's theorem applied to a strait, which becomes, when friction is taken into account,*

$$v^2 = \frac{2\,g\,(\zeta_{,} - \zeta_{,,})}{1 + \zeta' \frac{P_1}{\Omega_1} L_1 + \zeta' \frac{P_2}{\Omega_2} L_2 \left(\frac{\Omega_1}{\Omega_2}\right)^2 + \zeta' \frac{P_3}{\Omega_3} L_3 \left(\frac{\Omega_1}{\Omega_3}\right)^2 + \ldots}$$

in which $\zeta_{,}$ and $\zeta_{,,}$ denote the heights of the surface of the water, at the Gate and beyond, above half-tide level; ζ' a friction factor; P the wetted perimeter; Ω the area of the section, and L the length of the channel. The subscript, 1, refers to the Gate, and subscripts, 2, 3, etc., refer to other portions of the channel. The resistance, as expressed in the denominator, increases if the cross-section at the Gate, Ω_1, increases, the other sections, Ω_2, Ω_3, etc., remaining the same. The velocity then would vary inversely as Ω_1. Because the other sections, Ω_2, Ω_3, do not remain the same for both streams, the real velocity will vary less than changes in Ω_1.

Taking into account the information thus briefly outlined, for the whole length of the river, it "seems probable that the volume transmitted westerly can be not more than one or two per cent. greater than the volume transmitted easterly."†

As the volume flowing on each tidal current is about 4 000 000 000 cu. ft., the net or resultant flow from the Sound to the Upper Bay would be, at 2%, 80 000 000 cu. ft. per tidal cycle.

East River at Brooklyn Bridge.—The section of the river at Brooklyn Bridge is 1 500 ft. wide, and its average depth below mean water level is 44.20 ft., so that its area is 66 300 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 2.42 knots, and on flood 2.22 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 2.32 knots, or 14 105 ft. per solar hour.

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* See Appendix C.

† Letter, U. S. Coast and Geodetic Survey, August 14th, 1908.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows and Hudson River. The mean cross-sectional velocity, therefore, is 10 504 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 696 500 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 4 330 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

On page 715 it is shown that there is a net discharge through the East River from the Sound into the Upper Bay, and that it is about 80 000 000 cu. ft. per tidal cycle. This figure includes the mean yearly discharge of land-water run-off, which amounts to about 3 400 000 cu. ft. per tidal cycle of 12 lunar hours. From what has been stated on page 708, the average ebb flow will be the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 4 370 000 000 cu. ft.
Average volume of flood current, 4 290 000 000 " "

East River off 11th Street.—The section of the river off East 11th Street, Manhattan, is 2 235 ft. wide and its average depth below mean water level is 36.13 ft., so that its area is 80 750 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.91 knots, and on flood 1.85 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.88 knots, or 11 430 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows and Hudson River. The mean cross-sectional velocity, therefore, is 8 572 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 692 200 000 cu. ft. per solar hour, as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 4 299 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

As shown on page 715, the net discharge toward the Upper Bay is about 80 000 000 cu. ft. per tidal cycle. The average volumes of ebb and flood currents were obtained in the same manner as those for the East River at Brooklyn Bridge.

Average volume of ebb current, 4 339 000 000 cu. ft.
Average volume of flood current, 4 259 000 000 " "

East River off 31st Street.—The section of the river off East 31st Street, Manhattan, is 3 180 ft. wide, and its average depth below mean water level is 39.07 ft., so that its area is 124 250 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.13 knots, and on flood 1.29 knots. These velocities were obtained by averaging a number of observations, made near this section by the U. S. Coast and Geodetic Survey, of the surface velocities at strength. It was assumed that the velocities vary as ordinates of a sinusoid, and the average velocities were obtained by multiplying the velocities at strength by $\frac{2}{\pi}$.

The mean of the ebb and flood average velocities of the swiftest surface thread is 1.21 knots, or 7 357 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows, Hudson River, and the other sections on the East River. The mean cross-sectional velocity, therefore, is 5 518 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 685 600 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 4 258 000 000 cu. ft. obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

As shown on page 715, the net discharge toward the Upper Bay is about 80 000 000 cu. ft. per tidal cycle. The average volumes of ebb and flood currents were obtained in the same manner as those for the East River at Brooklyn Bridge.

Average volume of ebb current, 4 298 000 000 cu. ft.
Average volume of flood current, 4 218 000 000 " "

*East River off 81st Street.**—The west channel is 800 ft. wide, and its average depth below mean water level is 39 ft., so that its area

* The data for this estimate are from a letter of the U. S. Coast and Geodetic Survey dated August 14th, 1908.

is 31 200 sq. ft. Similar figures for the east channel, a little southward from Graham Avenue are 600, 26.5 and 15 900, respectively. In the former section the maximum surface velocity is 4.9 knots and in the latter 4.0 knots, according to observations made in 1857 and 1874. Multiplying these velocities by 0.75 to reduce them to cross-sectional velocities at time of strength, and by $\frac{2}{\pi}$ to reduce them to mean cross-sectional velocities during each tide, and by the area of the section, the following volumes were obtained:

2 756 295 600 cu. ft. passing western section.
1 146 685 915 " " " eastern "

The sum of these two volumes is 3 902 981 515 cu. ft., which is the mean volume flowing on ebb and flood currents. Dividing this figure into the average volume flowing on ebb and on flood, in the same manner as for the East River off Brooklyn Bridge, the following figures were obtained:

Average volume of ebb current, 3 943 000 000 cu. ft.
Average volume of flood current, 3 863 000 000 " "

East River off Old Ferry Point.—The section of the river off Old Ferry Point is 3 000 ft. wide, and its average depth below mean water level is 43.88 ft., so that its area is 131 650 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 0.88 knot, and on flood 1.09 knots. These velocities were obtained by averaging a number of observations of the surface velocities at strength at this point, made by the U. S. Coast and Geodetic Survey. Assuming that the velocities vary as the ordinates of a sinusoid, the average surface velocity at strength was multiplied by $\frac{2}{\pi}$ to obtain the average velocity of the swiftest surface thread. The mean of the ebb and flood average velocities of the swiftest surface thread is 0.985 knot or 5 989 ft. per solar hour.

The coefficient to reduce the surface velocity to cross-sectional velocity was taken at 0.75, the same as for The Narrows, Hudson River, and the other sections on the East River. The mean cross-sectional velocity, therefore, is 4 491 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 591 300 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 3 672 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

As shown on page 715, the net discharge toward the Upper Bay is about 80 000 000 cu. ft. per tidal cycle. The average volumes of ebb and flood currents were obtained in the same manner as those for the East River at Brooklyn Bridge.

Average volume of ebb current (westerly), 3 712 000 000 cu. ft.
Average volume of flood current (easterly), 3 632 000 000 " "

Reason for the Volumes of Flow Diminishing from the Southerly End to the Easterly End of the East River.

The average volumes of flow for the ebb and flood currents at different sections of the East River, in cubic feet, are as follows:

	Ebb.	Flood.
Off Brooklyn Bridge..........	4 370 000 000	4 290 000 000
Off East 11th Street..........	4 339 000 000	4 259 000 000
Off East 31st Street..........	4 298 000 000	4 218 000 000
Off East 81st Street..........	3 943 000 000	3 863 000 000
Off Old Ferry Point..........	3 712 000 000	3 632 000 000

It will be seen from this statement that the volumes of flow decrease from the southerly end to the easterly end of the river. A reason for this decrease was sought, with the following result:

On Fig. 37 curves are drawn for each lunar hour, with distances as abscissas and velocities as ordinates, flood velocities being plotted above, and ebb velocities below, the datum lines. Another curve is plotted to the right showing the average elevation of the water in the East River for each lunar hour. This latter curve is cross-hatched for the times when the water is rising, and is left blank when falling.

It will be noted that, during the 6 lunar hours when the average level of the water is rising, practically for the entire length of the river, the current is flowing flood (north and east). In order that the level of the water may rise, it is necessary for more water to flow in at the Battery than flows out at Throgs Neck, in order to make up this rise. In the same way, for the 6 lunar hours when the average elevation of the water is falling, the current is ebb (flowing west and south) for practically the entire length of the stream.

In order that the elevation of the water may fall, it is necessary that a greater quantity should flow out of the river at the Battery than flows in at Throgs Neck.

In discussing this subject, it must be kept clearly in mind that the velocities of the currents are due to the heads produced by the varying hydraulic slopes, and that the rise and fall of the water are due to the progress of the tidal waves entering the East River from each end.

The advance of the tidal waves is retarded by the shallowness of the waters and by the natural obstructions. The tidal wave entering the East River at Governors Island is about 3 hours, 5 min. ahead of the wave entering at Throgs Neck. As these waves progress through the river in opposite directions, there is a superposition and a consequent interference. This interference and the natural obstructions of the river have the effect of making the wave vary from a true sinusoid.

As the Governors Island wave enters the river before the Throgs Neck wave, greater quantities of water have to flow past points on the lower reaches of the river than points farther up (northerly and easterly) so as to provide for the tidal prism, for, if not, there would be some point in the river (near its mid-length) where the water would not rise or fall.

Kill van Kull.

Kill van Kull off Port Richmond.—The section of the river off Port Richmond is 1 380 ft. wide, and its average depth below mean water level is 28.30 ft., so that its area is 39 060 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.48 knots, and on flood 1.24 knots. These velocities were obtained by plotting the results from the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.36 knots, or 8 269 ft. per solar hour.

The coefficient to reduce the surface velocity to cross-sectional velocity was taken at 0.75, the same as for The Narrows, Hudson River, and East River. The mean cross-sectional velocity, therefore, is, 6 202 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 242 200 000 cu. ft., as the mean rate of flow through the section.

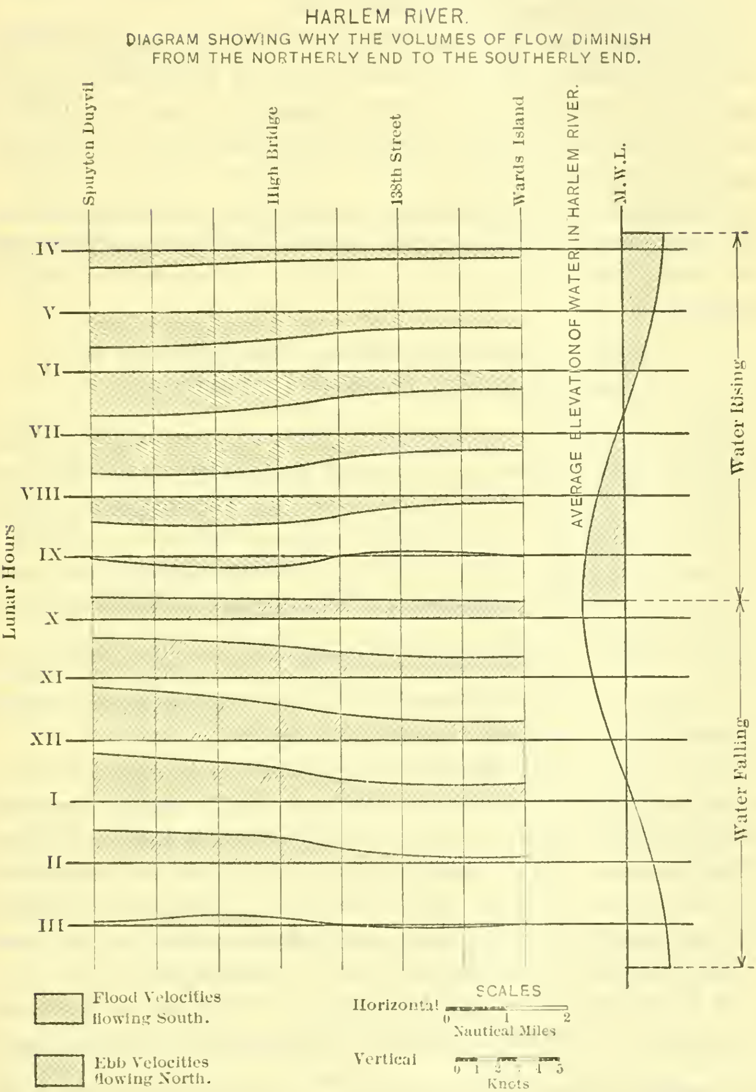

HARLEM RIVER.
DIAGRAM SHOWING WHY THE VOLUMES OF FLOW DIMINISH FROM THE NORTHERLY END TO THE SOUTHERLY END.
Spuyten Duyvil
High Bridge
138th Street
Wards Island
AVERAGE ELEVATION OF WATER IN HARLEM RIVER.
M.W.L.
IV
V
VI
VII
VIII
IX
X
XI
XII
I
II
III
Lunar Hours
Water Rising
Water Falling
Flood Velocities flowing South.
Ebb Velocities flowing North.
SCALES
Horizontal
0 1 2
Nautical Miles
Vertical
0 1 2 3 4 5
Knots

FIG. 37.

The mean volume flowing on ebb and flood currents is 1 502 000 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

The mean yearly discharge of land-water run-off through Kill van Kull for 6 lunar hours is 44 100 000 cu. ft., or 88 200 000 cu. ft. per tidal cycle of 12 lunar hours. These figures are from the records of the U. S. Coast and Geodetic Survey and the New York State Engineer and Surveyor. See Table 4.

From what has been stated on page 708, the average ebb volume will be the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of this net discharge, or:

Average volume of ebb current, 1 546 100 000 cu. ft.
Average volume of flood current, 1 457 900 000 " "

*Kill van Kull Between Constable Point and New Brighton.**—The section is 1 425 ft. wide, and its average depth below mean water level is 26 ft., so that its area is 38 475 sq. ft. The maximum surface velocity for this section is 2.3 knots at time of strength, according to observations made in 1856 and in 1885. Multiplying these velocities by 0.75 to reduce them to cross-sectional velocities at time of strength, and by $\frac{2}{\pi}$ to reduce them to mean cross-sectional velocities during each tide, and by the area of the section, the mean volume flowing through the section was obtained, namely, 1 595 475 341 cu. ft.

As a check on this figure, the U. S. Coast and Geodetic Survey* made the following calculation: The area of the waters above this section, including most of Kill van Kull, Newark Bay, the Passaic River as far as the Falls, and the Hackensack River and branches, is 15.3 sq. statute miles, or 426 539 520 sq. ft. The level of Newark Bay and near-by branches rises and falls about 4.5 ft. The tidal volume, therefore, is $426\,539\,520 \times 4.5 = 1\,919\,427\,840$ cu. ft.

A few observations in the Arthur Kill off Elizabethport, just above the mouth of the Elizabeth River, in 1856 and 1885, indicate a maximum surface velocity of 1.8 knots, or $\frac{18}{23}$ of the value observed in the

* These calculations are from a letter of the U. S. Coast and Geodetic Survey, dated August 14th, 1908.

Kill van Kull. The section of the Arthur Kill is about $600 \times 16 =$ 9 600 sq. ft., which would be equivalent to $\frac{18}{23}$ of 9 600, or 7 513 sq. ft., if the velocity were 2.3 knots, instead of 1.8 knots. Hence, the proportion of the volume flowing through the mouth of the Kill van Kull would be

$$\frac{38\,475}{38\,475 + 7\,513} = 0.8366313.*$$

This proportion, 0.8366313, multiplied by 1 919 427 840, gives 1 605 853 409 cu. ft. for the mean volume entering and leaving Kill van Kull. This agrees well with the results from the determination first given.

Averaging the two figures given by the U. S. Coast and Geodetic Survey for the flow of water through Kill van Kull between Constable Point and New Brighton, 1 600 000 000 cu. ft. is obtained as the mean volume flowing on ebb and flood currents. This figure was divided into volumes flowing on ebb current and flood current in the same manner as for Kill van Kull off Port Richmond, using 88 200 000 cu. ft. per tidal cycle for the mean yearly discharge of land-water run-off.

Average volume of ebb current, 1 644 100 000 cu. ft.
Average volume of flood current, 1 555 900 000 " "

The tidal prism (or the area multiplied by the mean range of tide) between the section off Port Richmond and the section from Constable Point to New Brighton is about 100 000 000 cu. ft.

The mean volume flowing through the section off Port Richmond would be less than the quantity flowing past the section at Constable Point by the tidal prism between these two places, as that much water would be used in increasing the level of the water. Subtracting the tidal prism from the flow through the section, 1 600 000 000 — 100 000 000 = 1 500 000 000 cu. ft. This figure agrees well with the mean volume flowing on ebb and flood currents, 1 502 000 000, as calculated for Kill van Kull, off Port Richmond, as stated on page 722.

Harlem River.

Harlem River off 144th Street.—The section of the river off West 144th Street, Manhattan, is 460 ft. wide and its average depth below mean water level is 15.67 ft., so that its area is 7 210 sq. ft.

* The proportion flowing through the Arthur Kill would be 0.1633687.

The average velocity of the swiftest surface thread during the tidal period on ebb is 0.71 knot, and on flood 0.69 knot. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 0.70 knot, or 4 256 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows, Hudson River, East River, and Kill van Kull. The mean cross-sectional velocity, therefore, is 3 192 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 23 010 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 142 900 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

At 144th Street the average water level during ebb (northerly) current is 0.98 ft. above mean water level, and during flood (southerly) current is 0.98 ft. below mean water level. The average area of cross-section during ebb current, therefore, is the mean area of the section plus the width multiplied by 0.98, or 7 660 sq. ft.; and the average area of cross-section during flood current is the mean area of the section minus the width multiplied by 0.98, or 6 760 sq. ft. The mean volume flowing on ebb and flood was divided in proportion to the average area of cross-sections on the ebb current and on the flood current.

Average volume of ebb current, 151 800 000 cu. ft.
Average volume of flood current, 134 000 000 " "

Harlem River 600 Feet North of High Bridge.—The section of the river 600 ft. north of High Bridge is 410 ft. wide, and its average depth below mean water level is 11.71 ft., so that its area is 4 800 sq. ft.

The average velocity of the swiftest surface thread during the tidal period on ebb is 1.30 knots, and on flood 1.22 knots. These velocities were obtained by plotting the results of the float observations of the Metropolitan Sewerage Commission, in the same manner as those for The Narrows. The mean of the ebb and flood average velocities of the swiftest surface thread is 1.26 knots, or 7 661 ft. per solar hour.

The coefficient to reduce the surface velocities to cross-sectional velocities was taken at 0.75, the same as for The Narrows, Hudson

River, East River, and Kill van Kull. The mean cross-sectional velocity, therefore, is 5 746 ft. per solar hour. Multiplying the area of the section by the mean cross-sectional velocity gives 27 580 000 cu. ft. as the mean rate of flow through the section.

The mean volume flowing on ebb and flood currents is 171 200 000 cu. ft., obtained by multiplying the rate of flow through the section by the length of time, or 6.21 solar hours.

At this section the average water level during ebb (northerly) current is 0.58 ft. above mean water level, and during flood (southerly) current is 0.58 ft. below mean water level. The average area of cross-section during ebb current, therefore, is the mean area of the section plus the width multiplied by 0.58, or 5 040 sq. ft.; and the average area of cross-section during flood current is the mean area of the section minus the width multiplied by 0.58, or 4 560 sq. ft. The mean volume flowing on flood and ebb was divided in proportion to the average area of cross-sections on the ebb current and on the flood current.

Average volume of ebb current, 179 800 000 cu. ft.
Average volume of flood current, 162 600 000 " "

Reason for the Volumes of Flow Diminishing from the Northerly End to the Southerly End of the Harlem River.

Although it was not possible to calculate accurately the flow of water in the Harlem River at places other than 144th Street and 600 ft. north of High Bridge, because enough float observations were not made at other points, it is apparent that the quantity of water flowing past any section decreases from the northerly to the southerly end.

The float observations by the Metropolitan Sewerage Commission near Wards Island, in the Harlem River, show small velocities and a tendency of the floats to eddy and drift toward the shore.

The few float observations in the Harlem River near Spuyten Duyvil show velocities considerably greater than those at the southerly end.

Fig. 38 was made to show why the volumes of flow diminish from the northerly to the southerly end. This diagram is similar in construction to Fig. 37 for the East River. The distance from Spuyten Duyvil to Wards Island is laid off for the lunar hours as abscissas and the velocities as ordinates. The flood velocities, or those flowing south,

are laid off to scale below the datum lines, and the ebb velocities, or those flowing north, are laid off to scale above the datum lines.

A curve is drawn at the left to show the average elevation of the water in the Harlem River at each lunar hour. This curve is cross-hatched for the times when the water is rising, and is left blank when falling.

It will be noted that, during the 6 lunar hours when the average level of the water is rising, the current is flowing flood for the entire length of the river. In order that the level of the water may rise, it is necessary that a greater quantity should flow in through Spuyten Duyvil than flows out at Wards Island, to make up this increase of elevation.

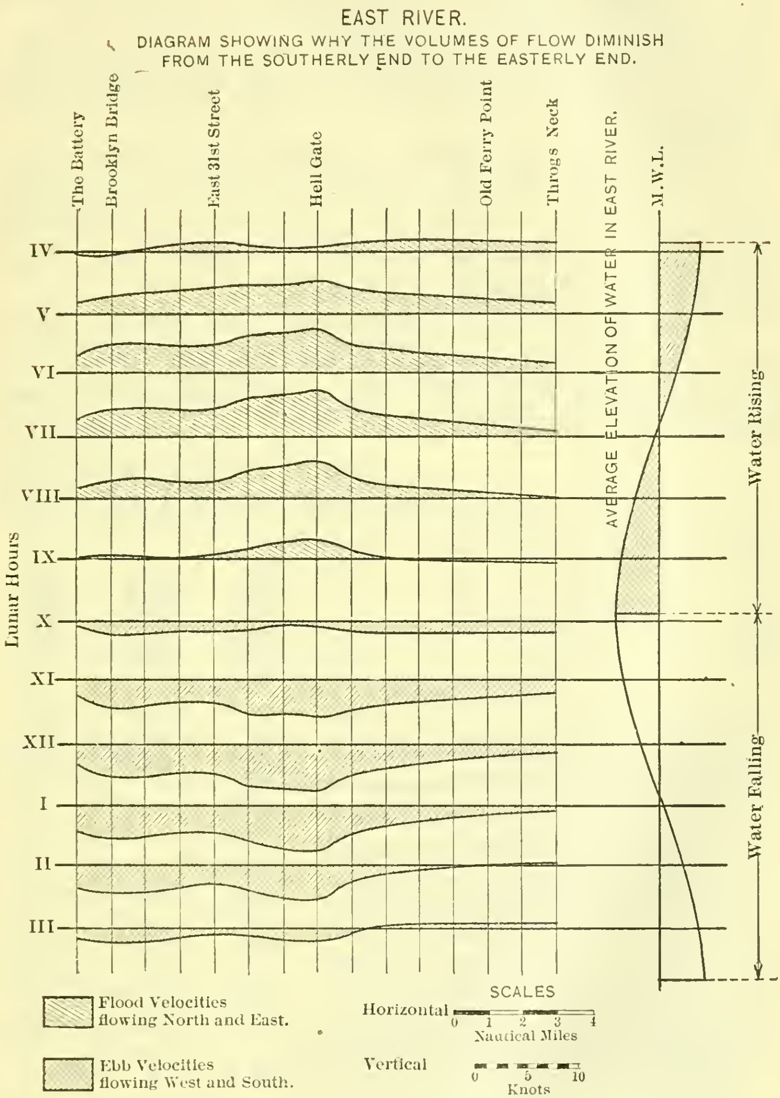
In the same way, for the 6 lunar hours during which the average level of the water is falling, the current is flowing ebb for the entire length of the river. In order that the level of the water may fall, it is necessary that more water flow out through Spuyten Duyvil than flows in at Wards Island. If the same quantities should pass Wards Island and Spuyten Duyvil during these 6 lunar hours, the average elevation of the water in the river would not fall, but would remain practically constant. The converse is also true. This is the reason that the volumes of flow diminish from the northerly to the southerly end of the Harlem River.

Resultant Flows.

The volumes flowing on ebb currents exceed those flowing on flood currents, so that there is a resultant flow during each tidal period through the harbor toward the sea.

In the rivers discharging into the harbor, the mean resultant flow equals the mean fresh-water run-off from the water-sheds above. When the ranges (rise and fall) of two successive tides are equal, a quantity equal to the fresh-water run-off must flow out and not return, so as to keep the elevation of the water the same on the next tide. As the ranges change from tide to tide, this will not be true in each individual instance, but will be true under mean conditions.

The following resultant flows are true only for mean conditions; this is, when the range of tide is normal and the mean quantity of land-water run-off is flowing. They are the difference between the flood and ebb discharges, as shown in the preceding paragraphs.



EAST RIVER.
DIAGRAM SHOWING WHY THE VOLUMES OF FLOW DIMINISH FROM THE SOUTHERLY END TO THE EASTERLY END.
The Battery
Brooklyn Bridge
East 31st Street
Hell Gate
Old Ferry Point
Throgs Neck
AVERAGE ELEVATION OF WATER IN EAST RIVER.
M.W.L.
IV
V
VI
VII
VIII
IX
X
XI
XII
I
II
III
Lunar Hours
Water Rising
Water Falling
Flood Velocities flowing North and East.
Ebb Velocities flowing West and South.
SCALES
Horizontal
0 1 2 3 4
Nautical Miles
Vertical
0 5 10
Knots

FIG. 38.

The Narrows.—The resultant flow through The Narrows is made up of the resultant flows of the Hudson River, the East River, and the Kill van Kull. It amounts to 1 262 400 000 cu. ft. per tidal cycle.

Hudson River.—The resultant flow in the Hudson River is the fresh-water run-off from the water-sheds above, namely, 1 087 200 000 cu. ft. per tidal cycle.

East River.—The resultant flow in the East River is small, and is caused probably by the average cross-sectional area being slightly greater when the water is flowing ebb (south) than when it is flowing flood (north). As shown on page 715, the resultant flow (south) amounts to about 80 000 000 cu. ft. per tidal cycle.

The Kill van Kull.—The resultant flow in the Kill van Kull is made up of its proportionate share of the fresh-water run-off of the rivers tributary to Newark Bay. By proportionate share is meant 83.66313% of the total. The other 16.33687% flows out through the

TABLE 7.—Estimates of Tidal Volumes of Ebb and Flood Currents,

No.	Elements of Problem.	The Narrows between Forts Wadsworth and Hamilton.	Hudson River off W. 39th St., Manhattan.	East River off Brooklyn Bridge.
(1)	Width of section, in feet	5 280	3 900	1 500
(2)	Average depth below M. W. Level, in feet	61.0	37.44	44.20
(3)	Area of section, in square feet	322 000	146 000	66 300
(4)	Average, velocity of swiftest surface thread: Ebb, in knots	1.34	1.86	2.42
(5)	Average, velocity of swiftest surface thread: Flood, in knots	1.17	1.26	2.22
(6)	Average, velocity of swiftest surface thread: Mean, in knots	1.255	1.56	2.32
(7)	Average, velocity of swiftest surface thread: Mean, in feet per solar hour	7 631.0	9 485	14 105
(8)	Coef., to reduce (7) to cross-sectional velocity	0.75	0.75	0.75
(9)	Mean cross-sectional velocity, in feet per solar hour	5 723	7 113	10 504
(10)	Mean rate of flow through section, in millions of cubic feet per solar hour (3) × (9)	1 837	1 038	696.5
(11)	Number of solar hours in 6 lunar hours	6.21	6.21	6.21
(12)	Mean volume flowing on ebb and flood currents, in millions of cubic feet (10) × (11)	11 410	6 447	4 330
(13)	Mean yearly discharge of land-water run-off per 6 lunar hours, in millions of cubic feet	591.2 } 631.2	543.6	
	Resultant from East River, 80 million cubic feet	40.0 }		80
	Average volume flowing through section, in millions of cubic feet per tide:			
(14)	On ebb	12 041.2	6 990.6	4 370
(15)	On flood	10 778.8	5 903.4	4 290
(16)	Resultant flow, excess of ebb over flood, in millions of cubic feet (14) — (15)	1 262.4	1 087.2	80

Arthur Kill (page 722). The resultant flow through the Kill van Kull amounts to 88 200 000 cu. ft. per tidal cycle.

Harlem River.—The resultant flow in the Harlem River is probably due to the greater average cross-sectional area which obtains on the ebb (northerly flow) than when the flow is in the reverse direction. Calculations gave 17 800 000 cu. ft. off 144th Street, and 17 200 000 cu. ft. at a section 600 ft. north of High Bridge. The mean resultant flow throughout the length of the river is about 17 400 000 cu. ft. per tidal cycle.

Under conditions of maximum net outflow toward the sea, the resultant flow is a little more than twice that under normal conditions. Under conditions of minimum net outflow, the resultant flow is about one-half of that under normal conditions.

In Table 7 are given the various steps by which the ebb and flood discharges, and the resultant flows, were calculated. This table is a

Discharges, in Cubic Feet.
Harbor of New York.

East River off E. 11th St., Manhattan.	East River off E. 31st St., Manhattan.	East River off E. 81st St., Manhattan.	East River off Old Ferry Point.	The Kill van Kull off Port Richmond	The Kill van Kull between Constable Pt. and New Brighton.	Harlem River off 144th St., Manhattan.	Harlem River Section 600 ft. North of High Bridge.
2 235 36.13 80 750	3 180 39.07 124 250	U. S. C. and G. Survey Letter of August 14th, 1908.	3 000 43.88 131 650	1 380 28.30 39 060	1 425 26 38 475	460 15.67 7 210	410 11.71 4 800
1.91	1.13		0.88	1.48	U. S. C. and G. Survey Letter of August 14th, 1908.	0.71	1.30
1.85	1.29		1.09	1.24		0.69	1.22
1.88	1.21		0.985	1.36		0.70	1.26
11 430 0.75	7 357 0.75		5 989 0.75	8 269 0.75		4 256 0.75	7 661 0.75
8 572	5 518		4 491	6 202		3 192	5 746
692.2 6.21	685.6 6.21		591.3 6.21	242.2 6.21		23.01 6.21	27.58 6.21
4 299	4 258	3 903	3 672	1 502	1 600	142.9	171.2
				44.1	44.1	Dividing (12) *pro rata* to average areas during currents gives (14) and (15).	
80	80	80	80				
4 339 4 259	4 298 4 218	3 943 3 863	3 712 3 632	1 546.1 1 457.9	1 644.1 1 555.9	151.8 134.0	179.8 162.6
80	80	80	80	88.2	88.2	17.8	17.2

summary of the matter contained under the heading, "Volumes of Tidal Discharges," pages 710 to 723.

It is self-evident that, under the conditions which obtain for the East and Harlem Rivers, especially the former, there may be great variations from day to day in the resultant flow as given in the preceding paragraphs. It is possible that in the East River the resultant flow on some days may be many times greater than that stated for the normal, and on other days it is possible that it is in the opposite direction, or northerly and easterly. These variations are brought about by various causes, the principal one being the wind inducing abnormal tides in the Upper Bay and in Long Island Sound.

Volumes of Tidal Flow Into and Out of the Upper Bay.

Taking the Upper Bay as a central basin, the entrances and exits are the East River, Hudson River, Kill van Kull, and The Narrows. Into the Upper Bay there is a discharge of water from the Hudson and a proportion of the discharge of the Hackensack and Passaic Rivers, as well as the resultant flow from the Sound through the East River. This resultant flow through the East River toward the Upper Bay may or may not be great, but, under normal conditions, there is a small resultant discharge into the Bay. In consequence, there is a net flow of water seaward through The Narrows, and the effect of this seaward flow is felt possibly some 60 miles off Sandy Hook.*

While the water is flowing into and out of the Upper Bay during a full tidal cycle, the volume of water in the Upper Bay is changing, as is shown by the rise and fall of its water surface. In order to create this rise, more water must flow into the Upper Bay during approximately 6 lunar hours than runs out during the same hours. The converse also is true. As the water surface of the Upper Bay is nearly 21 sq. miles, and the mean range of tide is 4.4 ft., the tidal prism and also the volume of tidal change between any two periods of time can be estimated.

The rates or volumes of flow, in millions of cubic feet per lunar hour, are shown graphically in Fig. 8. To determine the volumes of flow through any one of the entrances to the Upper Bay at any given lunar hour, measure the ordinate to the proper curve in accordance

* U. S. Coast and Geodetic Survey, Vol. VII, 1859-60, Appendix 26.

with the scale marked at the side of the figure. These rates of flow are those under mean conditions, and are sufficiently accurate for all practical purposes.

Fig. 8 was constructed by estimating that the rates of flow vary as the ordinates of a curve of sines. Therefore, for each tidal flow, a sine curve was drawn to scale, so that its length would represent the number of lunar hours that the current would flow on either ebb or flood; and its area, between the curve and the base line, would represent the volumes of flow transmitted, as estimated under the heading, "Volumes of Tidal Discharges," page 710. Dividing any volume of tidal discharge by six will give the average flow for one lunar hour. Multiplying this result by $\pi \div 2$, which is the ratio that the maximum ordinate of a sine curve bears to its average ordinate, will give the maximum flow for the current selected. This point was plotted to scale, and a sine curve was drawn through it. This work was repeated for each branch of the harbor.

In drawing the curves on Fig. 8 the lunar times were advanced by the number of minutes that the tidal waves required to progress from the Upper Bay to the sections selected for estimating the volumes of flow.

The rates of flow, in millions of cubic feet per lunar hour, were calculated, and the results are given in Table 9 for each branch of the harbor. The results for each lunar hour were obtained by multiplying the maximum flow for the current selected, as determined previously, by the cosine of the angle, which represents in degrees the difference between the lunar time of maximum strength and the lunar time for which the calculation was made.

By way of illustration, the calculation for the first lunar hour is given in Table 8.

The figures in Table 9 are the calculated rates of flow at each lunar hour. The rates of flow are not the volumes flowing throughout a full lunar hour, although they approximate to the latter. In consequence, the columns add up only approximately to the ebb and flood discharges, as given in the preceding paragraphs.

The quantity of water flowing in at any lunar hour should equal that flowing out, when allowance is made for the tidal prism in the Upper Bay. The accuracy of the work is shown by taking the algebraic sum of the ordinates in Fig. 8, which should be zero,

or by adding the horizontal lines at each lunar hour in Table 9, which should give equal quantities of water flowing into and out of the Upper Bay. The error is small, considering the assumptions made, in view of the lack of actual current observations extending over long periods of time.

TABLE 8.—For Lunar Hour I.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Lunar time of maximum strength, water flowing into Upper Bay.	Lunar hours before I, lunar time.	Equivalent, in degrees.	Cosine of Column 4.	Volume flowing, in millions of cubic feet on ebb tide.	Maximum rate of flow, in millions of cubic feet per lunar hour on ebb. $\frac{\text{Column 6}}{6} \times \frac{\pi}{2}$	Rate of flow, in millions of cubic feet per lunar hour at 1 o'clock, lunar time. = Column 5 × Column 7
The Narrows	VI. 7	6.3	189	− 0.9877	12 041	3 153	− 3 114
Hudson River	I. 8	11.2	336	+ 0.9135	6 991	1 830	+ 1 672
East River	XII. 7	0.3	9	+ 0.9877	4 370	1 143	+ 1 129
Kill van Kull	X. 8	2.2	66	+ 0.4067	1 644	430	+ 175
Increase or decrease of water in Upper Bay	X. 8	2.2	66	+ 0.4067	2 530	663	+ 270
							+ 3 246
							− 3 114
						Error	132

In Column 8 the minus sign indicates that the water is flowing out of, and the plus sign that it is flowing into, the Upper Bay.

TABLE 9.—Rates of Flow, in Millions of Cubic Feet per Lunar Hour.

Lunar Hours.	The Narrows.		Hudson River.		East River.		Kill van Kull.		Water in Upper Bay.	
	Into Upper Bay.	Out of Upper Bay.	Into Upper Bay.	Out of Upper Bay.	Into Upper Bay.	Out of Upper Bay.	Into Upper Bay.	Out of Upper Bay.	Decreasing.	Increasing.
I	....	3 114	1 672	....	1 129	....	175	....	270	....
II	....	2 450	1 820	....	888	....	....	43	....	69
III	....	1 130	1 480	....	410	....	....	240	....	390
IV	441	....	744	....	....	176	....	372	....	606
V	1 775	....	....	162	....	707	....	406	....	659
VI	2 634	....	....	909	....	1 048	....	330	....	536
VII	2 786	....	....	1 412	....	1 109	....	166	....	270
VIII	2 192	....	....	1 537	....	873	45	....	69	....
IX	1 011	....	....	1 251	....	402	253	....	390	....
X	....	493	....	629	179	....	393	....	606	....
XI	....	1 984	191	....	719	....	428	....	659	....
XII	....	2 944	1 076	....	1 067	....	348	....	536	....

Division of Sea- and Land-Water.

The salinity of the water in different parts of the harbor was observed by the Metropolitan Sewerage Commission from September, 1908, until December, 1909, inclusive. Eleven observation stations were established, as shown in Fig. 1. At each station observers were trained during the last months of 1908, and a complete record was made for every day of the year, 1909. Samples of water were taken each day at three different times, usually at 8.00 A. M., 12.00 Noon, and 4.00 P. M.

The term "land-water," as used in this investigation, represents any water between fresh-water and water containing approximately 300 parts of chlorine per 1 000 000 parts. Between these limits the observers would record the water as being 100% of land-water. The standard for "sea-water" was a water containing a salinity of 18 000 parts of chlorine per 1 000 000 parts, assuming that the chlorides represent 88.6% of the total specific gravity of sea-water. Under this condition, the salinometer reading would be 1.025 when corrected to a standard temperature of 60° Fahr.

The instruments were all calibrated, and, as a further check, their readings were compared with results obtained by chemical analyses. The indicated chlorine and the actual chlorine, as found by analysis, agreed very closely.*

The quantity of land-water discharged into the harbor varies according to the season, being greatest during April and least during August. When the discharge of land-water is at a maximum, the salt-water of the sea is driven down the Hudson, so that the river at Tarrytown is practically fresh. When the discharge of land-water is at a minimum, the salt-water works its way up the Hudson as far as Poughkeepsie.

If no land-water entered the harbor, the water therein would be as salty as that of the sea. As the record shows that the harbor water is salty, but less salty than sea-water, it is evident that the former is a mixture of sea-water and land-water. It was considered that these salinometer records could be used in finding the proportions of sea- and land-water in the harbor waters, and a study was made of them for this purpose.

About 13 000 observations for salinity were made in the harbor during 1909. Additional observations were made at different depths

* Report, Metropolitan Sewerage Commission, April 30th, 1910, p. 519.

below the surface. These indicated that, usually, the water was slightly more salty as the depth from the surface increased, that the difference was small, and that the difference in salinity at the surface and at various depths depended on the currents at the place of observation. If the currents were irregular in character, the difference in salinity was very small.

These observations were tabulated. The monthly averages of land-water are given in Table 10 for the different stations. In this table, the figures in the columns headed "All" are the averages of all the observations taken during a month. The figures in the column headed "Out" and "In" are the averages of all the observations taken during inflowing or outflowing currents. Some observations were taken when the direction of the current was doubtful, as at slack water, and these are not included under either "In" or "Out," but under "All."

A study was made of the records of the Weather Bureau, U. S. Department of Agriculture, so as to compare the rainfall of 1909 with the normal. The monthly and yearly precipitation at Albany, Carmel, Wappinger Falls, West Point, and New York, were averaged. The results of these averages are given in the last line of Table 10, and show the percentage of precipitation above or below the normal for each month of 1909.

The volumes of water flowing on ebb and flood currents, for different parts of the harbor, are stated under the heading, "Volumes of Tidal Discharges." From these data the volumes flowing at intermediate places were estimated (see Appendix B). These volumes are given in Table 11.

Additional information regarding specific gravities and temperatures of the water is given in Appendix D, as recorded by the salinity observations made by the Metropolitan Sewerage Commission.

The Upper Bay and Hudson River.—A longitudinal section was taken from The Narrows, through the Upper Bay and Hudson River as far as Tarrytown, the distance between the places being laid off as abscissas in Fig. 39. At the places named, the mean tidal flows of ebb tide, in cubic feet, were laid off as ordinates, and a smooth curve was drawn through these points.

This mean quantity of water flowing on ebb tide was then divided into land- and sea-water, using the salinometer percentages given in

TABLE 10.—SUMMARY OF SALINOMETER RECORDS FOR 1909.

Percentage of Land-Water at the Surface. Corrected for Temperature 60 degrees.

	Jan.			Feb.			Mar.			Apr.			May.			June.			July.		
	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.
Ambrose Channel	1.3	.....	....	6.5	.....	....	10.2	.....	....	10.8	.....	.....	15.6	.....	....	13.0	.....	....	6.9	.....	....
West Bank	19.3	19.7	18.3	34.2	35.8	33.4	35.0	34.5	34.9	45.1	44.3	45.6	42.2	41.7	41.6	27.8	28.5	27.3	17.3	17.1	17.5
Fort Wadsworth	29.0	28.2	30.0	50.7	50.8	52.4	50.5	49.9	51.0	64.9	59.8	67.9	59.1	56.0	61.0	38.4	38.0	39.1	25.5	24.3	28.2
Robbins Reef	21.9	23.3	24.6	42.6	41.8	42.8	43.2	43.3	41.4	56.1	55.6	55.8	52.4	52.5	51.6	34.0	33.6	33.8	21.5	21.1	21.6
Governors Island	30.9	29.4	33.7	40.5	39.0	43.4	49.3	49.1	49.9	64.0	60.1	67.2	59.1	56.6	61.0	39.5	38.7	38.9	25.3	24.6	27.5
Blackwells Island	26.5	26.5	26.1	35.9	36.9	34.6	37.8	38.0	37.6	45.4	45.3	44.9	42.3	41.7	42.9	33.7	34.0	33.4	27.7	26.8	28.2
Great Beds	31.7	33.1	29.2	47.1	49.5	44.1	46.5	48.6	44.5	58.2	59.4	57.0	53.6	55.5	50.7	37.6	38.9	36.1	28.7	29.3	28.0
Passaic Light	65.8	63.8	67.3	82.2	81.0	84.0	83.4	83.5	83.3	90.6	90.3	90.9	86.3	85.7	86.8	67.2	66.3	67.8	51.0	50.0	51.8
Fort Washington Point	62.2	60.7	63.2	83.3	83.2	83.7	84.0	83.9	84.0	94.7	94.7	95.1	90.6	90.7	90.5	74.5	74.4	74.7	56.0	56.0	55.7
Tarrytown	84.6	84.8	84.5	97.2	97.4	97.0	98.6	98.6	98.7	100.0	100.0	100.0	99.9	99.8	99.9	93.1	93.4	92.8	80.2	80.4	80.0
Throgs Neck	15.9	15.8	16.0	19.3	19.8	18.7	20.8	21.2	20.7	23.8	23.5	24.2	24.3	24.0	24.5	21.3	21.4	21.2	18.1	18.2	18.0
Rainfall, percentage above or below normal.*	— 6%			+ 32%			— 28%			+ 61%			— 36%			+ 13%			— 53%		

	Aug.			Sept.			Oct.			Nov.			Dec.			Averages for Year.		
	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In.	All.	Out.	In
Ambrose Channel	7.2	.....	....	6.8	.....	....	6.0	.....	....	4.6	.....	.....	4.6	.....	.....	7.6	....	....
West Bank	18.4	17.5	19.4	16.2	16.6	16.0	15.7	15.4	16.0	15.2	14.7	15.7	17.5	17.4	17.9	25.2	24.8	24.1
Fort Wadsworth	29.5	28.7	30.2	28.3	24.8	32.3	22.2	21.5	25.5	22.7	22.2	23.0	25.3	23.6	27.3	37.2	34.8	37.5
Robbins Reef	21.4	21.6	21.3	18.3	18.3	18.6	19.6	19.3	20.4	19.7	19.7	19.5	22.3	21.9	22.8	31.2	30.4	30.4
Governors Island	24.9	23.8	26.0	22.2	19.7	24.9	24.7	22.8	27.0	25.1	24.1	26.4	30.0	28.1	32.5	36.4	34.6	37.0
Blackwells Island	25.5	26.5	24.4	24.1	24.3	23.8	25.6	25.2	26.2	24.2	21.5	23.8	25.7	25.6	25.8	31.9	31.9	31.9
Great Beds	29.6	31.4	27.1	26.2	26.4	25.9	23.9	24.5	23.4	24.9	25.0	24.9	23.6	23.2	23.9	35.8	37.2	34.5
Passaic Light	52.6	52.2	53.6	47.5	47.3	48.3	46.4	46.2	46.1	45.3	44.6	45.8	57.6	56.0	59.0	61.2	63.8	64.9
Fort Washington Point	51.8	52.4	50.6	48.8	49.0	48.3	50.4	49.5	50.8	49.4	49.7	48.4	53.7	53.0	54.4	66.3	66.3	65.1
Tarrytown	74.4	75.2	73.7	73.0	73.5	72.6	73.8	74.4	73.4	73.4	73.5	73.3	76.4	76.5	76.3	85.3	85.7	84.9
Throgs Neck	17.3	17.7	17.0	17.1	16.9	17.4	15.5	15.0	15.9	13.2	13.3	13.2	11.6	11.4	11.7	18.1	18.2	18.1
Rainfall, percentage above or below normal.*	+ 14%			— 11%			— 74%			— 47%			+ 9%			— 12%		

* Average of precipitation at Albany, Carmel, New York City, Wappinger Falls, and West Point, from climatological Report, U. S. Weather Bureau.

Throgs Neck: West currents = In.

"All" means average of Ebb, Flood and Slack Water observations.

"Out" means average of Ebb observations.

"In" means average of Flood observations.

Table 10, and a smooth curve was drawn through these points, allowance being made if the precipitation was above or below the annual normal. This gave the middle line in the diagram, dividing land-water from sea-water.

TABLE 11.—Volumes Flowing on Ebb and Flood Currents, Harbor of New York, in Millions of Cubic Feet.

Part of Harbor.	Yearly Means.	
	Ebb.	Flood.
The Narrows	12 041	10 779
Hudson River, off The Battery	7 430	6 343
" " " 39th Street	6 990	5 903
" " " Fort Washington Point	6 230	5 143
" " " Tarrytown	3 980	2 893
East River, off Brooklyn Bridge	4 370	4 290
" " " East 11th Street, Manhattan	4 339	4 259
" " " " 31st Street, "	4 298	4 218
" " " " 81st Street, "	3 943	3 863
" " " Old Ferry Point	3 712	3 632
Newark Bay	1 972	1 866

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER.
WHEN AVERAGE LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

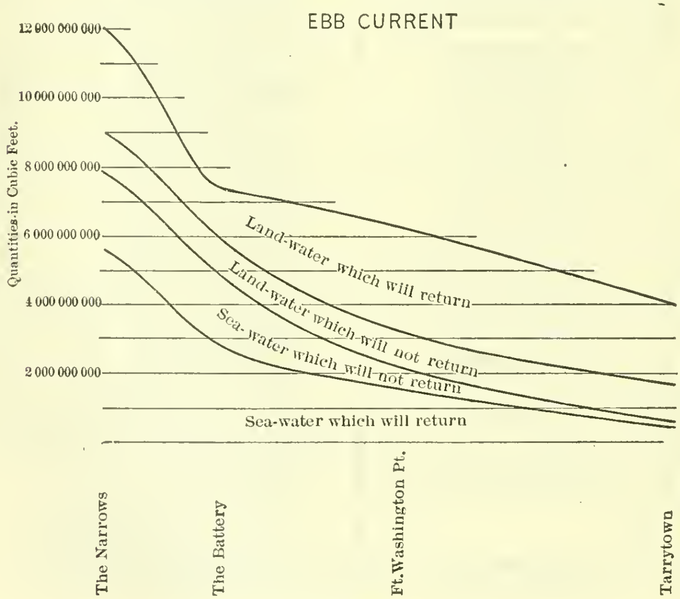


Fig. 39.

The land-water was then divided into "land-water which will return" and "land-water which will not return." This was done by plotting to scale at the different places the mean discharge of land-water for 12 lunar hours above the middle line. The remainder of the land-water is "land-water which will return." The discharge of land-water is the same as that given in Table 5. The land-water is held back by flood tide, so that the discharge for 12 hours has to pass out during ebb tide. The principle on which this division was made is, that if the density of the water in two successive ebb tides is to remain constant, the same quantity of land-water must flow out and not return on each ebb tide.

The sea-water was divided into "sea-water which will return" and "sea-water which will not return" by making the "sea-water which will not return" the same proportion of the total sea-water, as the "land-water which will not return" is of the total land-water. This is so because the land- and sea-waters are mixed, and the quantity of "land-water which will not return" will take with it its proportionate quantity of sea-water.

The division of sea- and land-water in a flood tide, when the yearly mean quantity of land-water is flowing, is shown in Fig. 40. This diagram was constructed by laying off the tidal flow of the mean flood, in cubic feet, as ordinates, and drawing a smooth curve through the points. This result is the same as subtracting from the top curve of Fig. 39, the quantity of "land-water which will not return." The "returning land-water" was obtained by laying off, from this top curve, the "land-water which will return," as given in Fig. 39. Below the second curve, the "sea-water which will return" was laid off, as given in Fig. 39, and marked "returning sea-water." The remaining part of the figure, the "new sea-water," is the same in quantity as the "sea-water which will not return," as given in Fig. 39.

In order to show the variation from mean conditions, Figs. 41, 42, 43, and 44 were constructed to show the division of sea- and land-water in ebb and flood tides during August and April, when the minimum and maximum quantities of land-water are flowing.

The division of sea- and land-water in ebb tide during August, when the minimum quantity of land-water is flowing, is shown in Fig. 41, which was constructed in a similar manner to Fig. 39, except that

the salinometer records for August, as given in Table 10, were used, and the quantity of land-water flowing from the water-sheds was taken for August, instead of the yearly mean.

The division of sea- and land-water in flood tide during August, when the minimum quantity of land-water is flowing, is shown in Fig. 42, which was constructed from Fig. 41, in a similar manner to Fig. 40.

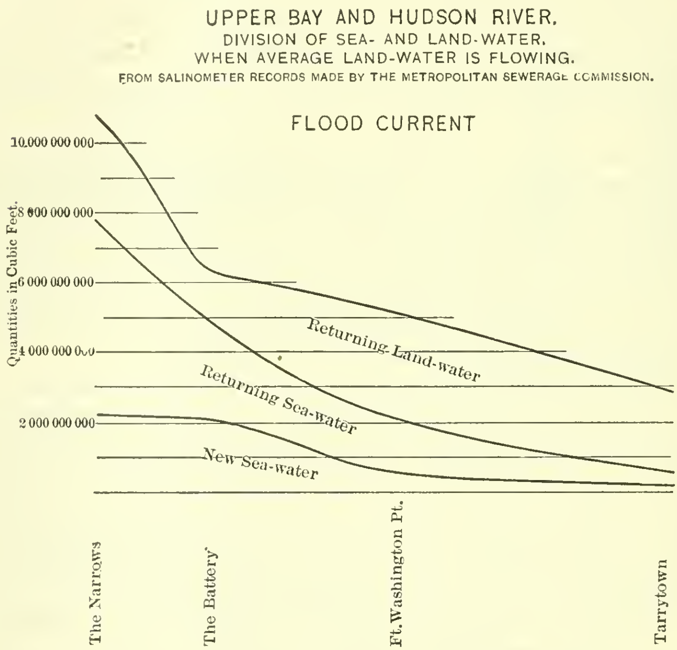


Fig. 40.

The division of sea- and land-water in ebb tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 43, which was constructed in a similar manner to Fig. 39, except that the salinometer records for April, as given in Table 10, were used, and the quantity of land-water flowing from the water-sheds was taken for April, instead of the yearly mean.

The division of sea- and land-water in flood tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 44, which was constructed from Fig. 43, in a similar manner to Fig. 40.

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER.
DURING AUGUST, WHEN MINIMUM LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

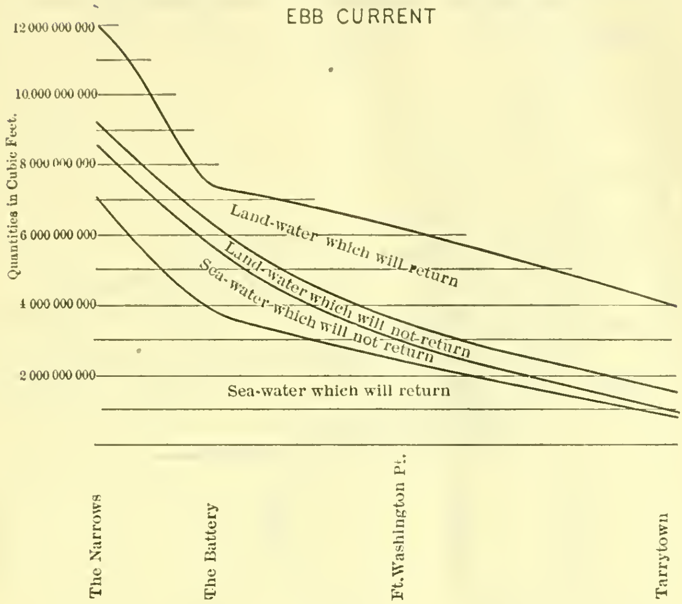


FIG. 41.

UPPER BAY AND HUDSON RIVER.
DIVISION OF SEA- AND LAND-WATER.
DURING AUGUST, WHEN MINIMUM LAND-WATER IS FLOWING.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

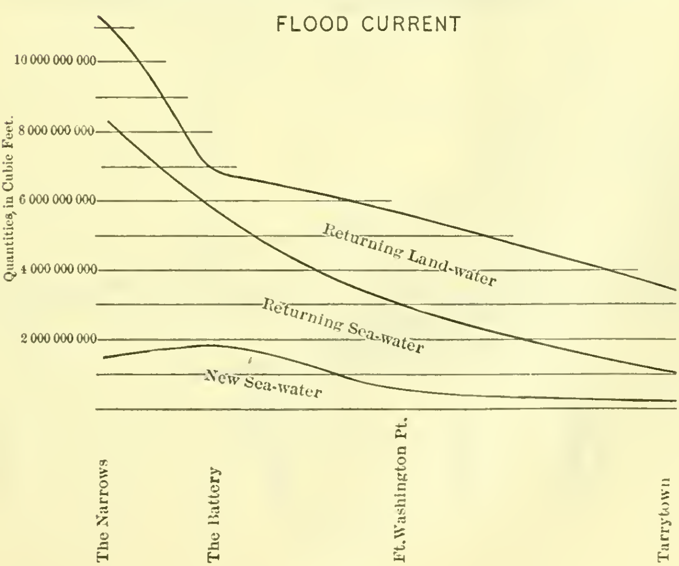


FIG. 42.

UPPER BAY AND HUDSON RIVER.

DIVISION OF SEA- AND LAND-WATER DURING APRIL, WHEN MAXIMUM LAND-WATER IS FLOWING.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

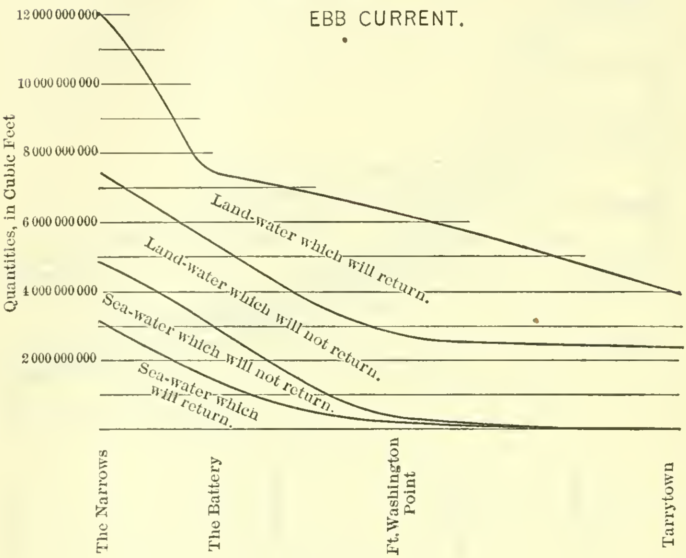


FIG. 43.

UPPER BAY AND HUDSON RIVER.

DIVISION OF SEA- AND LAND-WATER DURING APRIL, WHEN MAXIMUM LAND-WATER IS FLOWING.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

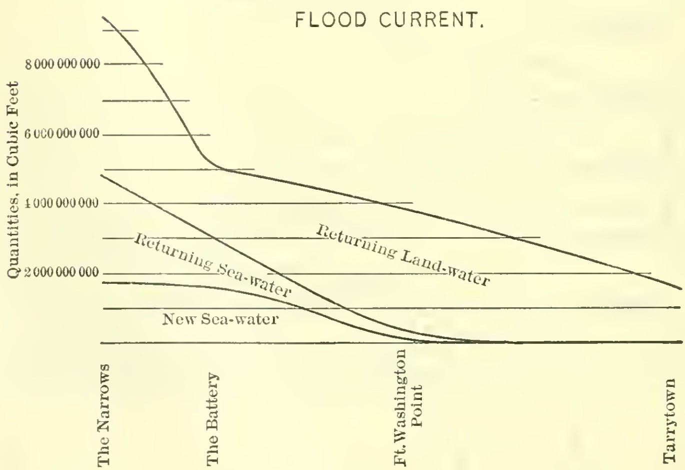


FIG. 44.

EAST RIVER.

DIVISION OF SEA- AND LAND-WATER WHEN AVERAGE LAND-WATER IS FLOWING.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

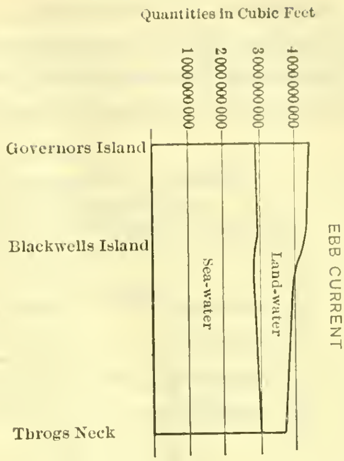


FIG. 45.

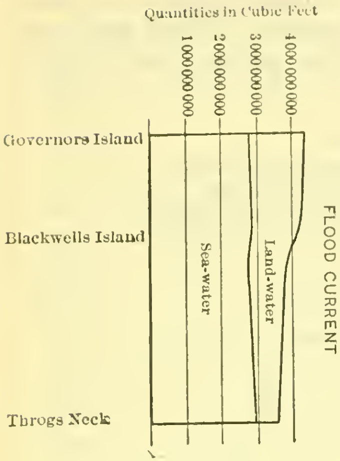


FIG. 46.

EAST RIVER.

DIVISION OF SEA- AND LAND-WATER DURING AUGUST, WHEN MINIMUM LAND-WATER IS FLOWING.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

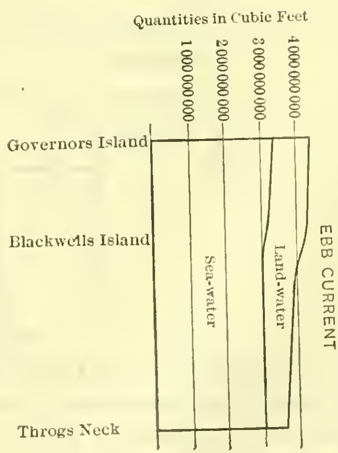


FIG. 47.

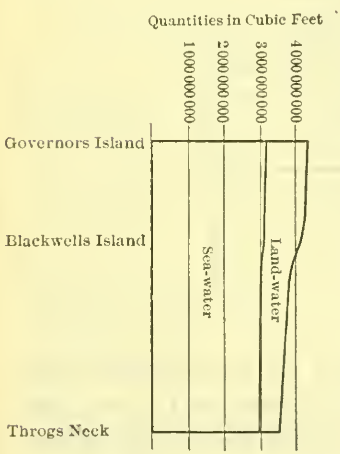


FIG. 48.

The East River.—In like manner, a longitudinal section was taken through the East River from Governors Island to Throgs Neck, and figures illustrating this section were drawn to the same scale as those for the Upper Bay and Hudson River.

EAST RIVER.

DIVISION OF SEA- AND LAND-WATER DURING APRIL, WHEN MAXIMUM LAND-WATER IS FLOWING.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

EBB CURRENT

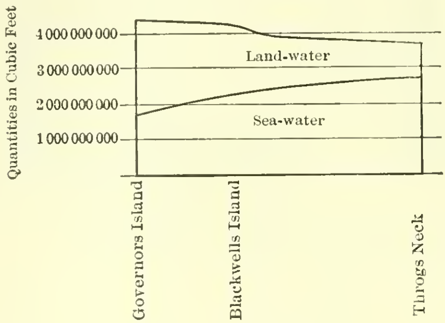


FIG. 49.

FLOOD CURRENT

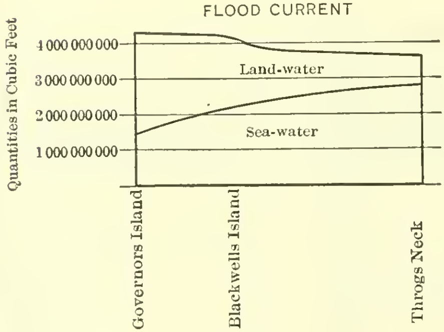


FIG. 50.

The division of sea- and land-water in ebb tide when the yearly mean quantity of land-water is flowing, is shown in Fig. 45. This diagram was constructed by laying off the mean flow, in cubic feet per tide, as ordinates, and dividing these ordinates according to the salinometer records, as given in Table 10, allowance being made if the precipitation was above or below the annual normal.

The division of sea- and land-water in flood tide when the yearly mean quantity of land-water is flowing, is shown in Fig. 46.

The division of sea- and land-water in ebb tide during August, when the minimum quantity of land-water is flowing, is shown in Fig. 47. The division of sea- and land-water in flood tide during August, when the minimum quantity of land-water is flowing, is shown in Fig. 48.

The division of sea- and land-water in ebb tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 49. The division of sea- and land-water in flood tide during April, when the maximum quantity of land-water is flowing, is shown in Fig. 50.

The results shown graphically in Figs. 39 to 50, inclusive, are given in Tables 12 to 17, inclusive. These tables also give the percentages which the division of sea- and land-water are of the total tidal flows.

TABLE 12.—Division of Sea- and Land-Water in Ebb Current when Yearly Average Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Ebb Current.

Part of Harbor.	Land-Water: which will return.	Land-Water: which will not return.	Sea-Water: which will return.	Sea-Water: which will not return.	Total ebb flow.
* Upper Bay and Hudson River.					
The Narrows	3 010	1 180	5 640	2 210	12 040
	25.0%	9.8%	46.8%	18.4%	100%
The Battery	1 480	1 090	2 800	2 060	7 430
	19.9%	14.7%	37.7%	27.7%	100%
Fort Washington Point	3 040	1 090	1 550	550	6 230
	48.8%	17.5%	24.9%	8.8%	100%
Tarrytown	2 320	1 090	390	180	3 980
	58.3%	27.4%	9.8%	4.5%	100%
† East River.					
Governors Island	1 520		2 870		4 390
	34.6%		65.4%		100%
Blackwells Island	1 320		2 830		4 150
	31.9%		68.1%		100%
Throgs Neck	660		3 000		3 660
	18.1%		81.9%		100%

* Shown graphically on Fig. 39.
† Shown graphically on Fig. 45.

Division of Sea- and Land-Water by Months.

A study was made of the division of sea- and land-water in both ebb and flood tides, to show the monthly variations at different places in the harbor.

TABLE 13.—Division of Sea- and Land-Water in Flood Current when Yearly Average Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Flood Current.

Part of Harbor.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
* Upper Bay and Hudson River.				
The Narrows	3 010	5 560 ‡	2 210	10 780
	27.9%	51.6%	20.5%	100%
The Battery	1 480	2 800	2 060	6 340
	23.3%	44.2%	32.5%	100%
Fort Washington Point	3 040	1 550	550	5 140
	59.1%	30.2%	10.7%	100%
Tarrytown	2 320	390	180	2 890
	80.3%	13.5%	6.2%	100%
† East River.				
Governors Island	1 590	2 720		4 310
	37.0%	63 0%		100%
Blackwells Island	1 300	2 770		4 070
	31.9%	68.1%		100%
Throgs Neck	650	2 930		3 580
	18.2%	81.8%		100%

* Shown graphically on Fig. 40. † Shown graphically on Fig. 46.
‡ This figure is 80 000 000 cu. ft. less than the "sea-water which will return," as given in Table 12, because this is the mean resultant flow through the East River.

TABLE 14.—Division of Sea- and Land-Water in Ebb Current, During August, when Minimum Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Ebb Current.

Part of Harbor.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
* Upper Bay and Hudson River.					
The Narrows	2 830	620	7 050	1 540	12 040
	23.5%	5.2%	58.5%	12.8%	100%
The Battery	1 200	570	3 830	1 830	7 430
	16.1%	7.7%	51.6%	24.6%	100%
Fort Washington Point	2 700	570	2 450	510	6 230
	43.3%	9.1%	39.3%	8.3%	100%
Tarrytown	2 420	570	800	190	3 980
	60.9%	14.3%	20.1%	4.7%	100%
† East River,					
Governors Island	1 040		3 350		4 390
	23.8%		76.2%		100%
Blackwells Island	1 100		3 050		4 150
	26.5%		73.5%		100%
Throgs Neck	620		3 040		3 660
	17.0%		83.0%		100%

* Shown graphically on Fig. 41. † Shown graphically on Fig. 47.

TABLE 15.—DIVISION OF SEA- AND LAND-WATER IN FLOOD CURRENT, DURING AUGUST, WHEN MINIMUM QUANTITY OF LAND-WATER IS FLOWING.

Figures in Millions of Cubic Feet and in Percentage During Flood Current.

Part of Harbor.	Land-Water returning.	Sea-Water. Returning.	Sea-Water. New.	Total flood flow.
*UPPER BAY AND HUDSON RIVER.				
The Narrows	2 830	6 970‡	1 540	11 340
	24.9%	61.5%	13.6%	100%
The Battery	1 200	3 830	1 830	6 860
	17.5%	55.8%	26.7%	100%
Fort Washington Point	2 700	2 450	510	5 660
	47.7%	43.3%	9.0%	100%
Tarrytown	2 420	800	190	3 410
	71.0%	23.4%	5.6%	100%
†EAST RIVER.				
Governors Island	1 120	3 190		4 310
	26.0%	74.0%		100%
Blackwells Island	990	3 080		4 070
	24.4%	75.6%		100%
Throgs Neck	630	2 950		3 580
	17.7%	82.3%		100%

* Shown graphically on Fig. 42. † Shown graphically on Fig. 48.
‡ This figure is 80 000 000 cu. ft. less than the "sea-water which will return," as given in Table 14. because this is the mean resultant flow through the East River.

TABLE 16.—DIVISION OF SEA- AND LAND-WATER IN EBB CURRENT, DURING APRIL, WHEN MAXIMUM QUANTITY OF LAND-WATER IS FLOWING.

Figures in Millions of Cubic Feet and in Percentage During Ebb Current.

Part of Harbor.	Land-Water: which will return.	Land-Water: which will not return.	Sea-Water: which will return.	Sea-Water: which will not return.	Total ebb flow.
*UPPER BAY AND HUDSON RIVER.					
The Narrows	4 610	2 590	3 100	1 740	12 040
	38.3%	21.5%	25.7%	14.5%	100%
The Battery	2 090	2 380	1 380	1 580	7 430
	28.1%	32.0%	18.6%	21.3%	100%
Fort Washington Point	3 520	2 380	200	130	6 230
	56.5%	38.2%	3.2%	2.1%	100%
Tarrytown	1 600	2 380	....	....	3 980
	40.2%	59.8%	....	....	100%
†EAST RIVER.					
Governors Island	2 640		1 750		4 390
	60.1%		39.9%		100%
Blackwells Island	1 880		2 270		4 150
	45.3%		54.7%		100%
Throgs Neck	890		2 770		3 660
	24.2%		75.8%		100%

* Shown graphically on Fig. 43. † Shown graphically on Fig. 49.

TABLE 17.—Division of Sea- and Land-Water in Flood Current, During April, when Maximum Quantity of Land-Water is Flowing.

Figures in Millions of Cubic Feet and in Percentage During Flood Current.

Part of Harbor.	Land-Water returning.	Sea-Water: Returning.	Sea-Water: New.	Total flood flow.
* Upper Bay and Hudson River.				
The Narrows	4 610	3 020 ‡	1 740	9 370
	49.2%	32.2%	18.6%	100%
The Battery	2 090	1 380	1 580	5 050
	41.4%	27.3%	31.3%	100%
Fort Washington Point	3 520	200	130	3 850
	91.4%	5.2%	3.4%	100%
Tarrytown	1 600	......	......	1 600
	100%	......	......	100%
† East River.				
Governors Island	2 900	1 410		4 310
	67.2%	32.8%		100%
Blackwells Island	1 830	2 240		4 070
	44.9%	55.1%		100%
Throgs Neck	840	2 740		3 580
	23.5%	76.5%		100%

* Shown graphically on Fig. 44.
† Shown graphically on Fig. 50.
‡ This figure is 80 000 000 cu. ft. less than the "sea water which will return," as given in Table 16, because this is the mean resultaut flow through the East River.

The Narrows.—The division of sea- and land-water in ebb tides by months is shown in Fig. 51. This diagram was constructed by laying off monthly divisions as abscissas, and the mean quantity of water flowing in ebb tide as ordinates. The curves dividing the "land-water which will return" and the "land-water which will not return" and the "sea-water which will return" and the "sea-water which will not return," were drawn in a similar manner to the curves shown on Fig. 39; except that the salinometer records were taken for each month from Table 10, instead of the mean for the year, and also the quantities of land-water flowing were taken for the different months, instead of the yearly mean.

The division of sea- and land-water in flood tides by months is shown in Fig. 52. This diagram was constructed by deducting from Fig. 51 the quantity of "land-water which will not return." This gives the upper curve of the diagram. The returning land-water was plotted by laying off, from this upper curve, the quantity of "land-water which will return," as given in Fig. 51. In a similar manner, the "returning

THE NARROWS.

DIVISION OF SEA- AND LAND-WATER BY MONTHS.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

EBB CURRENT.

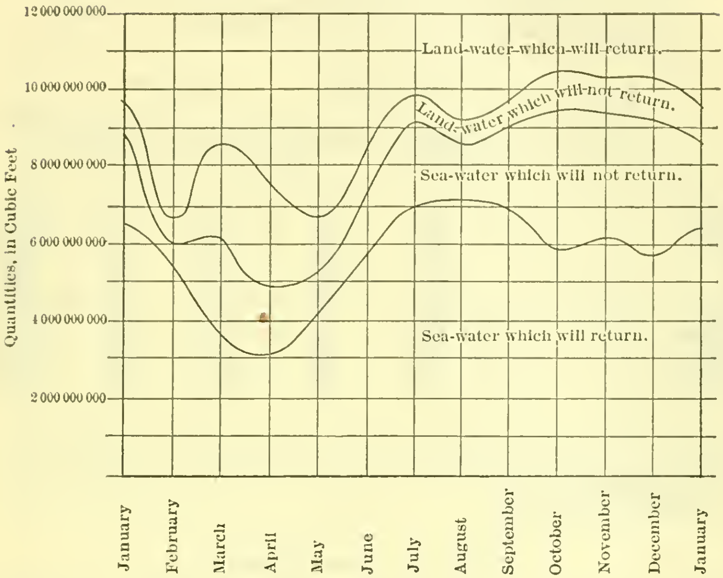


FIG. 51.

FLOOD CURRENT.

FIG. 52.

TABLE 18.—The Narrows.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water: which will return.	Land-Water: which will not return.	Sea-Water: which will return.	Sea-Water: which will not return.	Total ebb flow.
January	2 520	880	6 400	2 240	12 040
February	5 430	690	5 250	670	12 040
March	3 510	2 500	3 520	2 510	12 040
April	4 610	2 590	3 100	1 740	12 040
May	5 400	1 340	4 250	1 050	12 040
June	3 480	1 090	5 690	1 780	12 040
July	2 230	700	6 930	2 180	12 040
August	2 840	620	7 060	1 520	12 040
September	2 280	710	6 900	2 150	12 040
October	1 600	990	5 840	3 610	12 040
November	1 760	910	6 180	3 190	12 040
December	1 770	1 070	5 730	3 470	12 040
Average for Year†	3 010	1 180	5 640	2 210	12 040

* Shown graphically on Fig. 51.
† Average of daily readings.

TABLE 19.—The Narrows.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water: Returning.	Sea-Water: New.	Total flood flow.
January	2 520	6 320 ‡	2 240	11 080
February	5 430	5 170	670	11 270
March	3 510	3 440	2 510	9 460
April	4 610	3 020	1 740	9 370
May	5 400	4 170	1 050	10 620
June	3 480	5 610	1 780	10 870
July	2 230	6 850	2 180	11 260
August	2 840	6 980	1 520	11 340
September	2 280	6 820	2 150	11 250
October	1 600	5 760	3 610	10 970
November	1 760	6 100	3 190	11 050
December	1 770	5 650	3 470	10 890
Average for Year†	3 010	5 560	2 210	10 780

* Shown graphically on Fig. 52.
† Average of daily readings.
‡ These figures are 80 000 000 cu. ft. less than the "sea-water which will return," as given in Table 18, because this is the mean resultant flow through the East River.

HUDSON RIVER, OFF THE BATTERY.

DIVISION OF SEA- AND LAND-WATER BY MONTHS.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

EBB CURRENT.

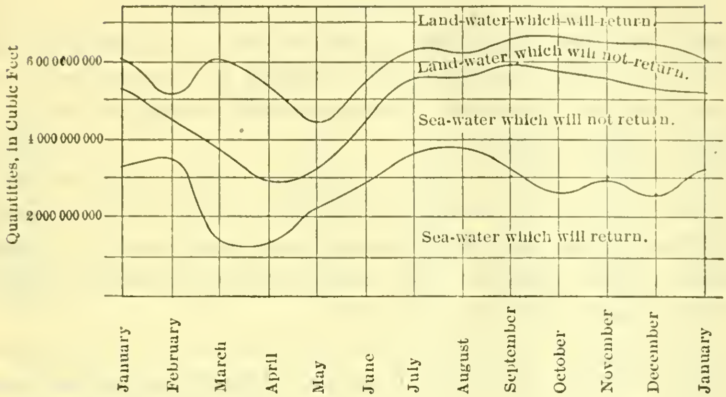


FIG. 53.

FLOOD CURRENT.

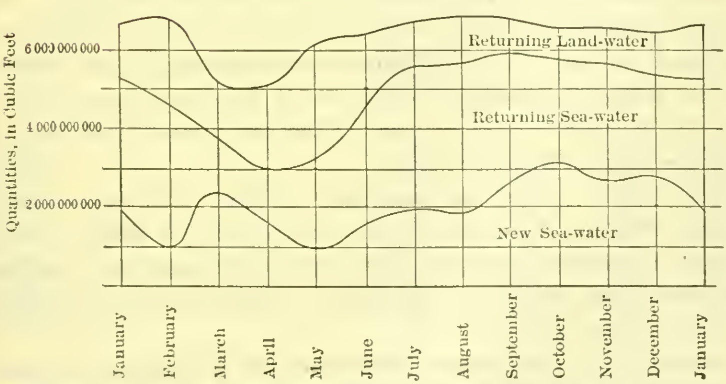


FIG. 54.

sea-water" was laid off, and the difference (the quantity to complete the diagram) is, therefore, "new sea-water," which is the same in quantity as the "sea-water which will not return," as given in Fig. 51. The results shown graphically in Figs. 51 and 52 are given in Tables 18 and 19.

The Hudson River, off the Battery.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 53, and in flood tides in Fig. 54. These diagrams were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 53 and 54 are given in Tables 20 and 21.

The Hudson River, off Fort Washington Point.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 55, and in flood tides in Fig. 56. These diagrams were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 55 and 56 are given in Tables 22 and 23.

The Hudson River, off Tarrytown.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 57, and in flood tides, in Fig. 58. These figures were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 57 and 58 are given in Tables 24 and 25.

The East River, off Blackwells Island.—The division of sea- and land-water in ebb tides, by months, is given in Fig. 59. This diagram was constructed by laying off monthly divisions as abscissas, and the average quantity of water flowing in ebb tide as ordinates. The average quantity of water flowing in ebb tide was then divided into land- and sea-water by the salinometer percentages for the different months, as given in Table 10, and a smooth curve was drawn through these points, allowance being made if the precipitation was above or below the normal for that month.

The division of sea- and land-water in flood tides, by months, is given in Fig. 60. This diagram was constructed in a similar manner to Fig. 59, excepting that the salinometer percentages for flood tide were used. The results shown graphically in Figs. 59 and 60 are given in Tables 26 and 27.

Newark Bay.—The figures illustrating the division of the waters in Newark Bay are not drawn to the same scale as the preceding ones. The scale was increased in order to show more clearly the division of the waters.

TABLE 20.—Hudson River, off the Battery.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water: which will return.	Land-Water: which will not return.	Sea Water: which will return.	Sea Water: which will not return.	Total ebb flow.
January	1 370	810	3 300	1 950	7 430
February	2 260	640	3 530	1 000	7 430
March	1 350	2 300	1 400	2 380	7 430
April	2 090	2 380	1 380	1 580	7 430
May	2 980	1 230	2 280	940	7 430
June	1 880	1 000	2 970	1 580	7 430
July	1 190	640	3 640	1 960	7 430
August	1 200	570	3 830	1 830	7 430
September	810	650	3 310	2 660	7 430
October	780	910	2 650	3 090	7 430
November	950	840	2 990	2 650	7 430
December	1 010	1 080	2 580	2 760	7 430
Average for Year†	1 480	1 090	2 800	2 060	7 430

* Shown graphically on Fig. 53.
† Average of daily readings.

TABLE 21.—Hudson River, off the Battery.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month	Land-Water returning.	Sea-Water: Returning.	Sea-Water: New.	Total flood flow.
January	1 370	3 300	1 950	6 620
February	2 260	3 530	1 000	6 790
March	1 350	1 400	2 380	5 130
April	2 090	1 380	1 580	5 050
May	2 980	2 280	910	6 200
June	1 880	2 970	1 580	6 430
July	1 190	3 640	1 960	6 790
August	1 200	3 830	1 830	6 860
September	810	3 310	2 660	6 780
October	780	2 650	3 090	6 520
November	950	2 990	2 650	6 590
December	1 010	2 580	2 760	6 350
Average for Year†	1 480	2 800	2 060	6 340

* Shown graphically on Fig. 54.
† Average of daily readings.

TABLE 22.—Hudson River, off Fort Washington Point.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water:		Sea-Water:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
January	2 970	810	1 930	520	6 230
February	4 540	640	920	130	6 230
March	2 930	2 300	560	440	6 230
April	3 520	2 380	200	130	6 230
May	4 420	1 230	450	130	6 230
June	3 640	1 000	1 250	340	6 230
July	2 850	640	2 240	500	6 230
August	2 700	570	2 450	510	6 230
September	2 400	650	2 500	680	6 230
October	2 170	910	2 220	930	6 230
November	2 260	840	2 280	850	6 230
December	2 220	1 080	1 970	960	6 230
Average for Year.†	3 040	1 090	1 550	550	6 230

* Shown graphically on Fig. 55.

† Average of daily readings.

TABLE 23.—Hudson River, off Fort Washington Point.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water:		Total flood flow.
		Returning.	New.	
January	2 970	1 930	520	5 420
February	4 540	920	130	5 590
March	2 930	560	440	3 930
April	3 520	200	130	3 850
May	4 420	450	130	5 000
June	3 640	1 250	340	5 230
July	2 850	2 240	500	5 590
August	2 700	2 450	510	5 660
September	2 400	2 500	680	5 580
October	2 170	2 200	930	5 300
November	2 260	2 280	850	5 390
December	2 220	1 970	960	5 150
Average for Year†	3 040	1 550	550	5 140

* Shown graphically on Fig. 56.

† Average of daily readings.

TABLE 24.—HUDSON RIVER, OFF TARRYTOWN.

DIVISION OF SEA- AND LAND-WATER IN EBB CURRENT, BY MONTHS.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	LAND-WATER:		SEA-WATER:		Total ebb flow.
	which will return.	which will not return.	which will return.	which will not return.	
January	2 570	810	460	140	3 980
February	3 240	640	80	20	3 980
March	1 620	2 300	25	35	3 980
April	1 600	2 380	....	....	3 980
May	2 740	1 230	7	3	3 980
June	2 720	1 000	190	70	3 980
July	2 560	640	620	160	3 980
August	2 420	570	800	190	3 980
September	2 280	650	820	230	3 980
October	2 050	910	710	310	3 980
November	2 090	840	750	300	3 980
December	1 970	1 080	600	330	3 980
Average for Year†	2 320	1 090	390	180	3 980

* Shown graphically on Fig. 57.

† Average of daily readings.

TABLE 25.—HUDSON RIVER, OFF TARRYTOWN.

DIVISION OF SEA- AND LAND-WATER IN FLOOD CURRENT, BY MONTHS.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	SEA-WATER:		Total flood flow.
		Returning.	New.	
January	2 570	460	140	3 170
February	3 240	80	20	3 340
March	1 620	25	35	1 680
April	1 600	...	...	1 600
May	2 740	7	3	2 750
June	2 720	190	70	2 980
July	2 560	620	160	3 340
August	2 420	800	190	3 410
September	2 280	820	230	3 330
October	2 050	710	310	3 070
November	2 090	750	300	3 140
December	1 970	600	330	2 900
Average for Year†	2 320	390	180	2 890

* Shown graphically on Fig. 58.

†Average of daily readings.

TABLE 26.—East River, off Blackwells Island.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water.	Sea-Water.	Total ebb flow.
January	1 100	3 050	4 150
February	1 530	2 620	4 150
March	1 580	2 570	4 150
April	1 880	2 270	4 150
May	1 730	2 420	4 150
June	1 410	2 740	4 150
July	1 110	3 040	4 150
August	1 100	3 050	4 150
September	1 010	3 140	4 150
October	1 050	3 100	4 150
November	1 020	3 130	4 150
December	1 060	3 090	4 150
Average for Year.†	1 320	2 830	4 150

* Shown graphically on Fig. 59.
† Average of daily readings.

TABLE 27.—East River, off Blackwells Island.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water.	Sea-Water.	Total flood flow.
January	1 060	3 010	4 070
February	1 410	2 660	4 070
March	1 530	2 540	4 070
April	1 830	2 240	4 070
May	1 750	2 320	4 070
June	1 360	2 710	4 070
July	1 150	2 920	4 070
August	990	3 080	4 070
September	970	3 100	4 070
October	1 070	3 000	4 070
November	970	3 100	4 070
December	1 050	3 020	4 070
Average for Year†	1 300	2 770	4 070

* Shown graphically on Fig. 60.
† Average of daily readings.

The division of sea- and land-water in ebb tides, by months, is shown in Fig. 61, and in flood tides, in Fig. 62. These figures were constructed in a similar manner to Figs. 51 and 52. The results shown graphically in Figs. 61 and 62 are given in Tables 28 and 29.

HUDSON RIVER, OFF FT. WASHINGTON POINT.

DIVISION OF SEA- AND LAND-WATER BY MONTHS.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

EBB CURRENT.

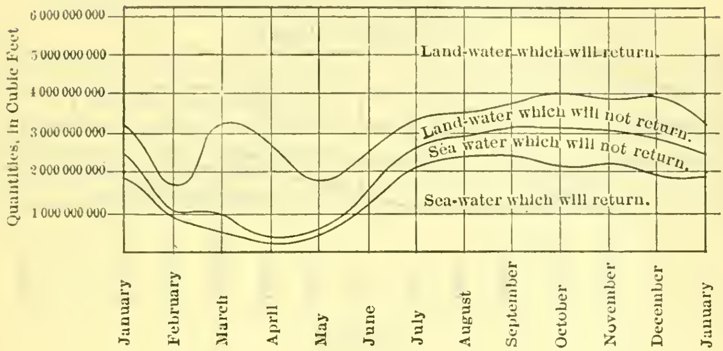


FIG. 55.

FLOOD CURRENT.

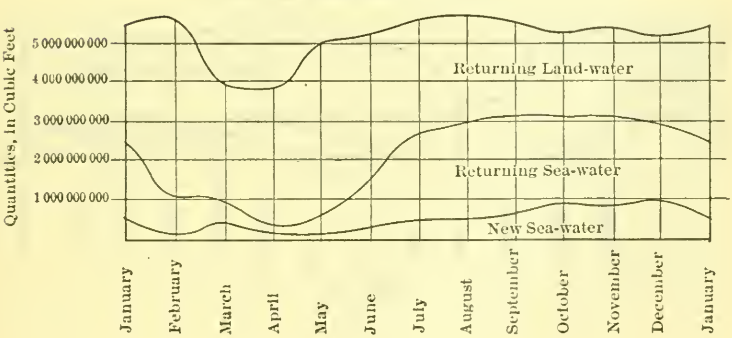


FIG. 56.

HUDSON RIVER, OFF TARRYTOWN

DIVISION OF SEA- AND LAND-WATER BY MONTHS.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

EBB CURRENT

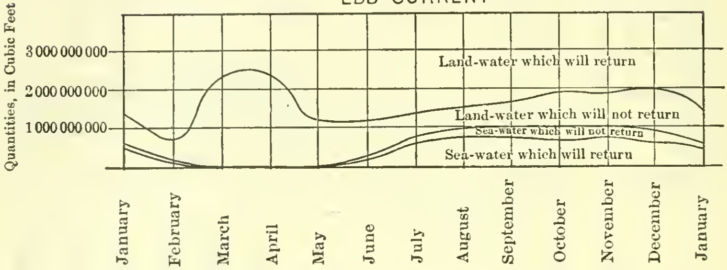


FIG. 57.

FLOOD CURRENT

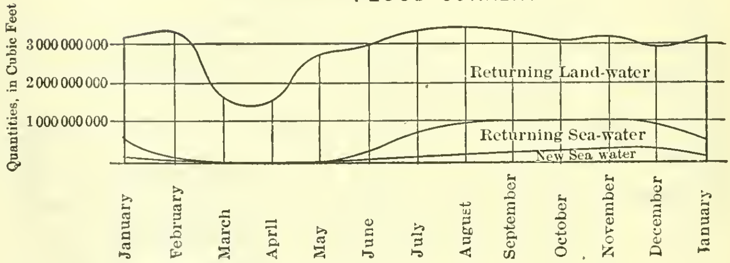


FIG. 58.

EAST RIVER, OFF BLACKWELL'S ISLAND

DIVISION OF SEA- AND LAND-WATER BY MONTHS.

FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.

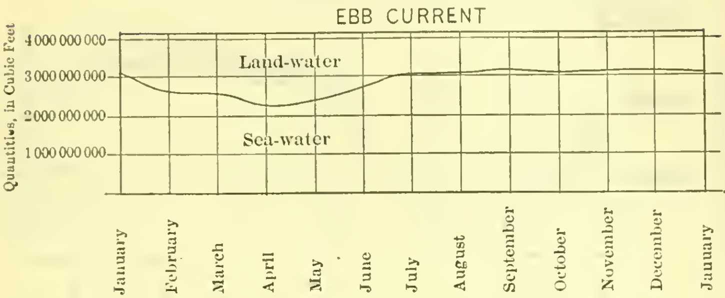


FIG. 59.

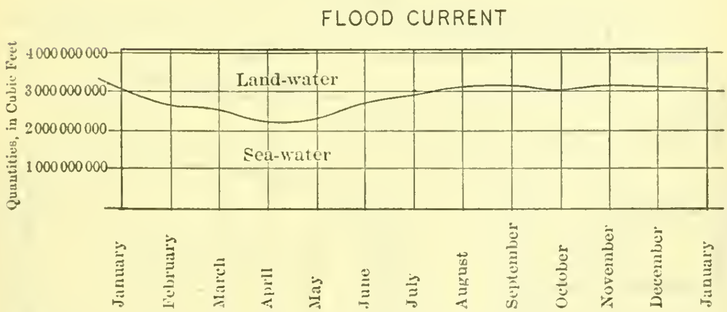


FIG. 60.

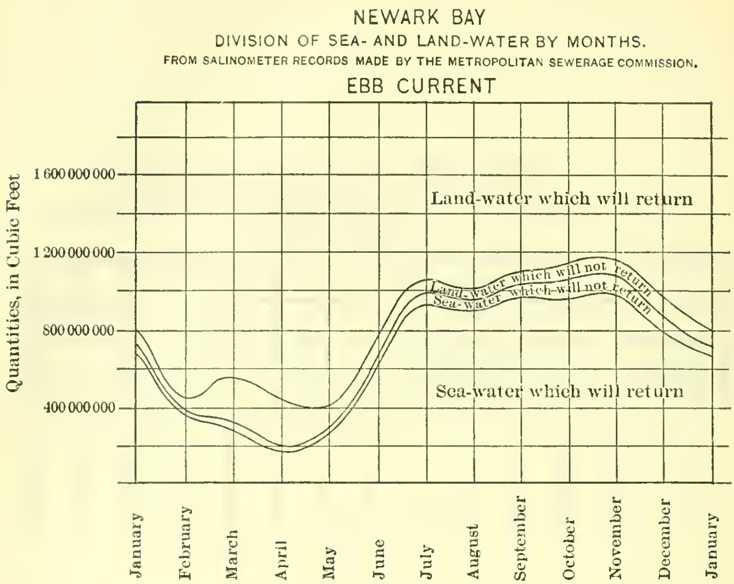

NEWARK BAY
DIVISION OF SEA- AND LAND-WATER BY MONTHS.
FROM SALINOMETER RECORDS MADE BY THE METROPOLITAN SEWERAGE COMMISSION.
EBB CURRENT
Quantities, in Cubic Feet
1 600 000 000
1 200 000 000
800 000 000
400 000 000
Land-water which will return
Land-water which will not return
Sea-water which will not return
Sea-water which will return
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 61.

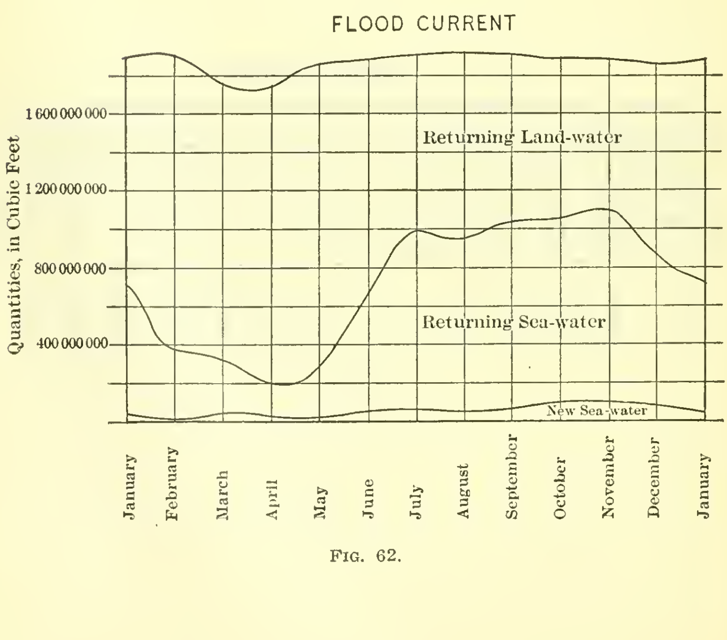

FLOOD CURRENT
Quantities, in Cubic Feet
1 600 000 000
1 200 000 000
800 000 000
400 000 000
Returning Land-water
Returning Sea-water
New Sea-water
January
February
March
April
May
June
July
August
September
October
November
December
January

FIG. 62.

TABLE 28.—Newark Bay.

Division of Sea- and Land-Water in Ebb Current, by Months.

Figures in Millions of Cubic Feet During Ebb Current.*

Month.	Land-Water: which will return.	Land-Water: which will not return.	Sea-Water: which will return.	Sea-Water: which will not return.	Total ebb flow.
January	1 179	79	669	45	1 972
February	1 535	62	360	15	1 972
March	1 424	223	281	44	1 972
April	1 549	231	167	25	1 972
May	1 570	120	262	20	1 972
June	1 210	97	616	49	1 972
July	924	62	924	62	1 972
August	973	56	892	51	1 972
September	870	63	969	70	1 972
October	823	88	959	102	1 972
November	799	81	992	100	1 972
December	999	105	785	83	1 972
Average for Year†	1 152	106	654	60	1 972

* Shown graphically on Fig. 61.
† Average of daily readings.

TABLE 29.—Newark Bay.

Division of Sea- and Land-Water in Flood Current, by Months.

Figures in Millions of Cubic Feet During Flood Current.*

Month.	Land-Water returning.	Sea-Water: Returning.	Sea-Water: New.	Total flood flow.
January	1 179	669	45	1 893
February	1 535	360	15	1 910
March	1 424	281	44	1 749
April	1 549	167	25	1 741
May	1 570	262	20	1 852
June	1 210	616	49	1 875
July	924	924	62	1 910
August	973	892	51	1 916
September	870	969	70	1 909
October	823	959	102	1 884
November	799	992	100	1 891
December	999	785	83	1 867
Average for Year†	1 165	644	57	1 866

* Shown graphically on Fig. 62.
† Average of daily readings.

Interpretation of Diagrams.

The sudden rise in the curve at the Battery, in Figs. 39, 40, 41, 42, 43, and 44, is caused by the discharge from and into the East River at the Battery, and also by the discharge from and into the Kill van Kull at Staten Island near The Narrows.

The salinometer readings at the station on Governors Island were used for the Battery, as it was the nearest observation station to this place. The salinometer readings at this station were affected more by the discharge from the East River than by that from the Hudson. In consequence, it is possible that more land-water should be shown at the Battery than in Figs. 39, 40, 41, 42, 43, and 44.

The diagrams showing the conditions existing in the East River, namely, Figs. 45, 46, 47, 48, 49, and 50, are not divided up like the others to show the "water which will return" and the "water which will not return." It was not possible to divide them in this manner, first, because the East River is a strait, open at both ends to bodies of salt-water, and second, because the water-sheds draining into it were so small that if a division had been attempted, as was done with the Hudson River, the "land-water which will not return" would have been so small that it would appear to be scarcely thicker than a line on the diagrams.

Conclusions.

A study of these diagrams shows that the flushing action of the waters of the harbor is not as great as the large volumes flowing on the tides would seem to warrant. All the diagrams show that the quantity of "sea- and land-water which will not return" is not a large proportion of the quantity flowing on an ebb tide. There is always a large quantity of "water which will return" on the succeeding flood tide.

The quantity of "new sea-water" is never a large proportion of the total quantity flowing on a flood tide. This "new sea-water" is only new off places situated near the ocean, and is not new at such places as the Upper Bay, Fort Washington Point, Tarrytown, East River, or Newark Bay. At these places the "new sea-water" is really water which has not been at the place on the previous tide, but comes from some other part of the harbor. For instance, at Fort Washington Point, the "new sea-water" would be from the lower reaches of the

Hudson, or Upper Bay, or perhaps some from the East River entering the Hudson at the Battery.

The water which has a seaward trend and will not return, is the quantity of land-water from the water-sheds. For places near the ocean, this seaward trend is increased by some "sea-water which will not return"; but, for places well within the harbor, such "sea-water which will not return" is replaced by sea-water (as explained previously) which comes from some other part of the harbor, and is not clean ocean-water.

The diagrams indicating the division of sea- and land-water by months show that the "sea-water which will return" is always greatest during the summer and least during the spring.

The "sea- and land-water which will not return" is least during February and the summer, and greatest during the spring.

The flushing out of the harbor, or the seaward trend, is greatest during the spring and least in February, under normal conditions.

Acknowledgments.

The writer acknowledges his thanks to David C. Johnson, Jun. Am. Soc. C. E., his office assistant, for aid in making the diagrams and calculations, and for numerous valued suggestions; also to O. H. Tittmann, M. Am. Soc. C. E., and Mr. F. W. Perkins, both of the U. S. Coast and Geodetic Survey, for their courtesies in permitting an inspection of the Survey records, and for their advice and criticisms.

APPENDIX A.

Harbor Divisions.

The boundaries for the several divisions of the harbor were selected as follows: At the entrance to the Lower Bay, by a line from Hook Beacon, on Sandy Hook, tangent to the westerly end of Rockaway Beach, to Barren Island. At the mouth of the Raritan River, by a line from Ferry Point to the extreme easterly point of South Amboy. At the entrance to the Shrewsbury River, by a line from the Post Light at Spermaceti Cove southwesterly to Highlands. At The Narrows, by a line drawn from the point at the south of Fort Hamilton Reservation to the dock about ½ mile south of Fort Wadsworth Light. At the south end of the East River, by a line drawn from the Battery to the foot of State Street, Brooklyn. At the south end of the Hudson River, from Castle Garden to the south end of the docks at Communipaw. At the east end of Kill van Kull, from Constable Point to the northerly point at New Brighton, Staten Island. At the west end of Kill van Kull, from the extreme end of Bergen Point southerly to Port Richmond. At the northerly end of Arthur Kill, by a line drawn from the railroad dock just east of Port Avenue, Elizabethport, southerly to Staten Island. At the south end of Arthur Kill, by a line drawn from Ferry Point to the extreme southwest end of Staten Island. At the upper end of Newark Bay, by lines drawn from the extreme southerly end of the Meadows, between the mouths of the Passaic and Hackensack Rivers, westerly across the Passaic and southeasterly across the Hackensack to the point of the Meadows. At the east end of the East River, by a line from the light on Throgs Neck to the northerly end of Willets Point. At the southerly end of the Harlem River, by a line from the foot of East 126th Street to the northerly corner of the Bronx Kills. At the northerly end of the Harlem River, by the New York Central and Hudson River Railroad bridge. The Bronx Kills, Little Hell Gate, the west branch between Manhattan and Randalls and Wards Islands, Newtown Creek, and Wallabout Bay are included in the East River. The areas of all islands have been subtracted.

APPENDIX B.

Calculation of Volumes Flowing on Tides.

Figures in Cubic Feet per Tidal Flow.

Hudson River, off the Battery:

The ebb flow equals the ebb off 39th Street plus the tidal prism between the Battery and 39th Street, Manhattan.

Ebb: 6 990 000 000 + 3.7 sq. miles × $(5\,280)^2$ × 4.3 ft., mean rise of tide	7 430 000 000
Mean discharge of fresh water in 12 lunar hours	1 087 000 000
Flood: Taken as the difference	6 343 000 000

Hudson River, off Fort Washington Point:

The ebb flow equals the ebb off 39th Street, minus the tidal prism between Fort Washington Point and 39th Street, Manhattan.

Ebb: 6 990 000 000 — 6.6 sq. miles × $(5\,280)^2$ × 4.15 ft., mean rise of tide	6 230 000 000
Mean discharge of fresh water in 12 lunar hours	1 087 000 000
Flood: Taken as the difference	5 143 000 000

Hudson River, off Tarrytown:

The ebb flow equals the ebb off Fort Washington Point, minus the tidal prism between Tarrytown and Fort Washington Point.

Ebb: 6 230 000 000 — 21.0 sq. miles × $(5\,280)^2$ × 3.85 ft., mean rise of tide	3 980 000 000
Mean discharge of fresh water in 12 lunar hours	1 087 000 000
Flood: Taken as the difference	2 893 000 000

Newark Bay:

The figures were taken from those of the Coast and Geodetic Survey, August 14th, 1908, as they contained an estimate of the flow through Arthur Kill.

The average of flood and ebb:

Through Kill van Kull	1 600 000 000
Through Arthur Kill	319 000 000
(1) Total, average for Newark Bay	1 919 000 000
(2) Mean fresh water discharged into Newark Bay during 6 lunar hours	52 757 000
Mean ebb flow, (1) + (2)	1 972 000 000
Mean flood flow, (1) — (2)	1 866 000 000

APPENDIX C.

Bernoulli's Theorem.

Bernoulli's theorem, applied to a strait connecting two large bodies of water, becomes, where friction is taken into account,

$$\frac{V_1^2}{2} = g\,(\zeta_, - \zeta_{,,}) - \zeta' \frac{P_1}{\Omega_1}\frac{V_1^2}{2} L_1 - \zeta' \frac{P_2}{\Omega_2}\frac{V_2^2}{2} L_2 - \ldots\ldots$$

Where V_1 = the velocity at the smallest section;

V_2, etc. = the velocities at other sections;

$\zeta_,$ and $\zeta_{,,}$ = the heights of the water each side of the smallest section;

ζ' = a friction factor;

P = the wetted perimeter;

Ω_1 = the area of the smallest cross-section;

Ω_2, etc. = areas of other cross-sections;

L_1 = the length of the channel for the smallest section;

L_2, etc. = lengths of other parts of the channel.

The conditions of continuity require, with the same volume flowing, the velocity to vary inversely as the cross-sections,

$$\frac{V_2}{V_1} = \frac{\Omega_1}{\Omega_2};\ \frac{V_3}{V_1} = \frac{\Omega_1}{\Omega_3};\ \text{etc.}$$

$$V_2 = V_1\left(\frac{\Omega_1}{\Omega_1}\right);\ V_3 = V_1\left(\frac{\Omega_1}{\Omega_3}\right).$$

Substituting in the first equation,

$$\frac{V_1^2}{2} = g\,(\zeta_, - \zeta_{,,}) - \zeta' \frac{P_1}{\Omega_1}\frac{V_1^2}{2} L_1 - \zeta' \frac{P_2}{\Omega_2}\frac{V_1^2}{2} L_2 \left(\frac{\Omega_1}{\Omega_2}\right)^2 - \ldots.$$

Solving for V_1,

$$V_1^2 = \frac{2\,g\,(\zeta_, - \zeta_{,,})}{1 + \zeta' \frac{P_1}{\Omega_1} L_1 + \zeta' \frac{P_2}{\Omega_2} L_2 \left(\frac{\Omega_1}{\Omega_2}\right)^2 + \ldots\ldots}$$

Note.—The value of ζ' is about 0.007, according to report of U. S. Coast and Geodetic Survey, 1907, Appendix 6, page 425.

APPENDIX D.

TABLE 30.—Specific Gravity and Temperatures. New York Harbor Waters.

Specific Gravity at 60° Fahr. Temperatures in Degrees, Fahrenheit.

	Ambrose Channel Light-Vessel.	West Bank Station.	Fort Wadsworth Station.	Robbins Reef Station.	Governors Island Station.	Blackwells Island Station.	Throgs Neck Station.	Fort Washington Point Station.	Tarrytown Station.	Passaic Light Station.	Great Beds Station.
January, 1909.											
Temperature, Max..................	45	44	43	43	41	44	40	41	37	38	41
" Min..................	38	34	30	35	32	32	30	27	33	30	30
Specific Gravity, Max..............	1.0256	1.0219	1.0196	1.0299	1.0299	1.0204	1.0219	1.0159	1.0069	1.0140	1.0204
" " Min..............	1.0239	1.0169	1.0145	1.0149	1.0199	1.0149	1.0209	1.0039	1.0014	1.0029	1.0094
" " Avg..............	1.02467	1.02021	1.01778	1.01873	1.01728	1.01838	1.02100	1.00951	1.00389	1.00854	1.01706
February, 1909.											
Temperature, Max..................	43	43	47	44	42	42	41	42	32	44	44
" Min..................	38	31	28	35	31	34	29	32	42	30	29
Specific Gravity, Max..............	1.0254	1.0214	1.0190	1.0209	1.0189	1.0199	1.0216	1.0129	1.0129	1.0134	1.0190
" " Min..............	1.0149	1.0089	1.0043	1.0059	1.0059	1.0059	1.0179	1.0000	1.0000	1.0009	1.0035
" " Avg..............	1.02312	1.00613	1.01196	1.01390	1.01478	1.01482	1.02005	1.00390	1.00650	1.00432	1.01296
March, 1909.											
Temperature, Max..................	44	42	43	45	43	49	43	45	42	50	45
" Min..................	38	32	30	39	30	35	35	32	33	30	33
Specific Gravity, Max..............	1.0250	1.0219	1.0193	1.0201	1.0159	1.0190	1.0209	1.0140	1.0000	1.0124	1.0175
" " Min..............	1.0164	1.0099	1.0089	1.0090	1.0070	1.0099	1.0179	1.0000	1.0034	1.0000	1.0084
" " Avg..............	1.02214	1.01625	1.01241	1.01428	1.01269	1.01555	1.01979	1.00406	1.00034	1.00118	1.01335
April, 1909.											
Temperature, Max..................	50	50	54	50	53	53	55	54	53	62	56
" Min..................	42	38	39	45	42	42	40	40	40	40	43
Specific Gravity, Max..............	1.0252	1.0191	1.0149	1.0161	1.0161	1.0181	1.0201	1.0052	1.0000	1.0063	1.0156
" " Min..............	1.0152	1.0088	1.0043	1.0075	1.0041	1.0082	1.0181	1.0000	1.0003	1.0000	1.0021
" " Avg..............	1.02229	1.01373	1.00878	1.01098	1.00899	1.01366	1.01904	1.00132	1.00000	1.00242	1.01044

TABLE 30. (*Continued.*)

	Ambrose Channel Light-Vessel.	West Bank Station.	Fort Wadsworth Station.	Robbins Reef Station.	Governors Island Station.	Blackwells Island. Station.	Throgs Neck Station.	Fort Washington Point Station.	Tarrytown Station.	Passaic Light Station.	Great Beds Station.
May, 1909.											
Temperature, Max.	62	59	65	62	63	60	63	70	62	65	65
" Min.	47	46	46	50	47	48	48	47	48	40	47
Specific Gravity, Max.	1.0243	1.0186	1.0154	1.0157	1.0147	1.0177	1.0197	1.0052	1.0000	1.0101	1.0173
" " Min.	1.0152	1.0094	1.0049	1.0080	1.0046	1.0102	1.0182	1.0005	1.0012	1.0000	1.0017
" " Avg.	1.02112	1.01203	1.01014	1.01144	1.01018	1.01440	1.01896	1.00230	1.00003	1.00339	1.01153
June, 1909.											
Temperature, Max.	70	70	74	71	71	71	71	80	71	82	72
" Min.	58	59	60	62	59	60	59	61	63	60	63
Specific Gravity, Max.	1.0240	1.0200	1.0169	1.0197	1.0177	1.0181	1.0248	1.0113	1.0038	1.0129	1.0176
" " Min.	1.0180	1.0155	1.0112	1.0123	1.0094	1.0145	1.0189	1.0031	1.0006	1.0032	1.0103
" " Avg.	1.02174	1.01800	1.01540	1.01650	1.01512	1.01656	1.01967	1.00636	1.00172	1.00822	1.01559
July, 1909.											
Temperature, Max.	72	72	74	72	75	71	73	79	75	82	78
" Min.	60	63	67	67	67	65	65	69	69	67	68
Specific Gravity, Max.	1.0245	1.0235	1.0216	1.0222	1.0205	1.0192	1.0215	1.0156	1.0068	1.0172	1.0194
" " Min.	1.0218	1.0180	1.0160	1.0156	1.0147	1.0157	1.0196	1.0057	1.0022	1.0075	1.0156
" " Avg.	1.02328	1.02069	1.01836	1.01960	1.01836	1.01805	1.02046	1.01098	1.00495	1.01226	1.01776
August, 1909.											
Temperature, Max.	73	72	75	74	74	73	74	77	75	78	76
" Min.	61	65	68	69	69	68	67	68	69	65	66
Specific Gravity, Max.	1.0241	1.0226	1.0216	1.0224	1.0205	1.0195	1.0216	1.0155	1.0090	1.0191	1.0201
" " Min.	1.0211	1.0182	1.0164	1.0167	1.0160	1.0179	1.0136	1.0083	1.0049	1.0044	1.0091
" " Avg.	1.02321	1.02041	1.01729	1.01963	1.01879	1.01862	1.02063	1.01245	1.00641	1.01189	1.01759

TABLE 30. (*Concluded.*)

	Ambrose Channel Light-Vessel.	West Bank Station.	Fort Wadsworth Station.	Robbins Reef Station.	Governors Island Station.	Blackwells Island Station.	Throgs Neck Station.	Fort Washington Point Station.	Tarrytown Station.	Passaic Light Station.	Great Beds Station.
September, 1909.											
Temperature, Max.	71	70	72	72	73	71	72	73	71	74	74
" Min.	64	64	61	64	62	67	63	64	63	61	64
Specific Gravity, Max.	1.0240	1.0230	1.0214	1.0249	1.0212	1.0210	1.0217	1 0174	1.0081	1.0205	1.0206
" " Min.	1.0221	1.0196	1.0160	1.0189	1.0171	1 0170	1.0194	1.0092	1.0049	1.0091	1.0166
" " Avg.	1.02314	1.02095	1 01791	1.02038	1.01945	1.01897	1.02073	1.01281	1.00674	1.01311	1.01841
October, 1909.											
Temperature, Max.	64	64	65	67	65	70	67	66	65	65	64
" Min.	51	49	48	52	51	51	49	48	46	44	46
Specific Gravity, Max.	1.0244	1.0232	1.0205	1.0250	1.0210	1.0205	1.0222	1.0154	1 0075	1.0193	1.0203
" " Min.	1.0224	1.0195	1.0159	1.0180	1.0157	1.0167	1.0195	1.0095	1.0051	1.0[illegible]88	1.0176
" " Avg.	1.02348	1.02106	1.01916	1.02009	1.01882	1.01859	1.02111	1.01238	1.00656	1.01340	1.01903
November, 1909.											
Temperature, Max.	55	52	53	54	55	56	56	55	51	55	51
" Min.	48	43	42	46	43	43	39	40	40	40	39
Specific Gravity, Max.	1.0249	1.0233	1.0198	1.0220	1.0206	1.0201	1.0230	1.0163	1.0088	1.0191	1.0196
" " Min.	1.0229	1.0202	1.0152	1.0184	1.0151	1.0173	1.0196	1.0092	1.0052	1.0091	1.0179
" " Avg.	1.02384	1.02120	1.01933	1.02007	1.01872	1.01896	1.02168	1.01266	1.00665	1.01367	1.01878
December, 1909.											
Temperature, Max.	49	43	43	45	45	46	46	42	41	44	40
" Min.	37	31	29	31	30	31	32	30	32	31	27
Specific Gravity, Max.	1.0246	1.0219	1.0194	1.0226	1.0205	1 0210	1.0230	1.0169	1.0079	1.0159	1.0224
" " Min.	1.0231	1.0189	1.0146	1.0159	1.0144	1.0159	1.0209	1.0081	1.0039	1.0024	1.0179
" " Avg.	1.02384	1.02361	1.01866	1.01940	1.01750	1.01992	1.02210	1.01111	1.00589	1.01053	1.01911

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THE PHILOSOPHY OF ENGINEERING.

By Maurice G. Parsons, Jun. Am. Soc. C. E.
To be Presented June 4th, 1913.

Introduction.

Notwithstanding that engineering structures have but recently come to be designed with precision, and that machinery as we know it to-day is a development of the past century, some of the greatest engineering feats were performed almost before history began. The Chinese wall, the Indian temples, the sphinx—the seven wonders of the world—will always compel our attention and baffle our imagination. In truth, the ancients were builders of no mean structures, but, except to some extent in irrigation, their real engineering ends there. They were not producers of economic necessities as we are to-day, but confined themselves to the erection of monuments, as such.

Later, during the Roman era, the need of political security, public utilities, and the spirit of commercialism came to be of more than monarchial whim, and resulted in the construction of highways, aqueducts, and sewers, which, again, command our admiration and respect, the more deservedly because they were for so long unsurpassed. Furthermore, it is remarkable that the next achievement, the reclamation work in Holland, is not overshadowed by an undertaking conforming more closely to these precedents, the fundamental cause for such a departure doubtless being economic necessity.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

All this time the engineer was, primarily, a builder without special training—with nothing but his judgment and common sense as guides. It remained for the Renaissance, with the steam engine, to produce the mechanic and the technical man busied with the construction of works of public utility, and for the present day to evolve what may be called the new engineer: the man of commerce, industry, business, who makes engineering a means rather than an end—the man of affairs.

Henceforth, managers and business men will more and more be chosen from among those engineers who can go back of stresses to the money side of things; and bankers are already relying on engineers to decide the advisability of investments, the kind and magnitude of structures, the operating and financial programme. For the engineer who can go still farther back—back of the dollar, to the public good—there opens up the vast field of government administration and policy forming—the field of the engineering economist—for government, particularly of municipalities, is in large measure of an engineering business nature. This is illustrated by appraisement work, especially, as is frequently the case, if it be in behalf of a political organization, where the appraiser must decide such questions as fair profit, what to do about franchise value, whether expenditures were justifiable, what class of service is adequate, and whether to insist on extending the right of eminent domain to public utility corporations.

Considering the opportunities newly presenting themselves to the engineer, there seem to be certain professional duties which we must discharge before we may walk worthy of the calling wherewith we are called, two of these being the obligations of contributing to education (of both the public and the Profession) and of applying newly established principles.

There should be no foundation, no possibility, for a remark that the engineer should be limited by the bounds of stresses and dimensions, and not stroll over into financiering and managing. Rather, it should be a matter of common report that the engineer, seeking the truth (as he must) stands not on the shows of things but on things themselves, that he, being closely related to all industrial adventures, must of necessity be familiar with questions governing their inception and their *modus operandi.* Over and above more generally establishing the engineer's fitness to sit in councils—literally based on this capability—is a higher duty, which will become more and more clearly

defined. This is the duty of reconciling business and the people, of being a peacemaker between those who produce our economic goods and those who consume them at what seems too high a cost. Many of the differences between capital and labor arise solely from misunderstandings, ill-selected points of view, and limited knowledge: to know all is to forgive all. There are a fortunate few who, in managing and evaluating, have opportunity to establish amicable relations between seller and buyer. There is a much greater number who can accomplish somewhat the same result by participating in professional discussions, local public meetings, and consultations of boards of directors, to the extent of explaining the complexities of modern sociology and commercialism.

To the end that members of the Profession may the more ably carry on this great work of public education, they should endeavor to educate one another by systematic and thorough interchange of ideas on such subjects as business and banking, human nature and government, rather than continue to leave each to strive as best each may, obtaining by individual effort a fragmentary knowledge thereof. To illustrate this point, it may not be out of place to call attention to the confusion existing in regard to "Going Value," "Water," and "Development Expenses," as brought out in the discussion of the paper by Henry Earle Riggs, M. Am. Soc. C. E., entitled "The Valuation of Public Service Property,"* and to the fact that no one seems to know just what to do with "discount on bonds," a question discussed in parts of that paper. Such a principle that good enough is best, that "one may get too much for one's money"† should be so established among engineers as to be applied habitually. Another example of the evil results of having centered perhaps too large a proportion of attention, as a Profession, on technical details, and too small a proportion on the working principles of big things, is the unfortunate statement that "it would be, not only bad engineering, but bad business,† as if engineering and business were not, fundamentally, one and the same thing—bad engineering can be nothing else than bad business.

The second large professional duty, that of taking the initiative of applying principles, is readily suggested by two statements‡ on

* *Transactions*, Am. Soc. C. E., Vol. LXXII, pp. 255, 276, and 187.

† "The Water-Works and Sewerage of Monterrey, N. L., Mexico," *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 581.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXII, pp. 246 and 282.

engineering and economics in the discussion on Mr. Riggs' paper, one to the effect that engineering occupies a part of the subject of economics, and the other that "the engineer is essentially an economist." Without quibbling over the question whether economics is a part of engineering or engineering a part of economics, it may be stated axiomatically that modern engineering is based, not on personal ambition, national pride, worship of stone, or military necessity, but on economics. A real political economist acts on this theory. Financiers and statesmen always have been such, and engineers now are. More than this, Courts at times require them to establish their theory. Our present social and commercial structure places on the engineer the obligation of taking the lead. The engineering economist, the new engineer, must guide the banker as well as build and operate his physical plants; he must act as mediator and policy former in questions of state; he must launch out into the deep. To quote from the Presidential Address of John A. Ockerson,* Past-President, Am. Soc. C. E.:

"There seems also to be a disposition to avoid participation in the discussion of public questions, even when closely related to the work of the Profession. When Congressional Committees call on the Society for advice with regard to pending legislation, involving questions relating to engineering, it would seem to be a proper function of the Society to render such aid as may be practicable.

"In fact, it might be well, under proper conditions, to go even farther, and use the influence the Society may have to mould public opinion along lines free from local or political bias, when our public works are the subject of discussion.

* * * * * * *

"As a matter of fact, the Society is already regarded by the public so highly that its members are looked on with special favor by the Courts when expert testimony is required, by the Government when seeking for capable men for service on public works, and by municipalities where men of integrity and ability are looked for to fill positions of trust relating to the engineering side of city government.

"In a speech at the Society House recently a leading politician of New York announced that they have come to realize that the interests of the city are best served by appointing engineers to fill the important offices which have charge of the physical welfare of the city in general.

"This is true of many of our cities, and the Profession is steadily growing in public favor; loyalty to the Society on the part of all its

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* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 1034.

members wherever they may be located will greatly stimulate this growth."

Led by his interest in this newly conceived field of professional activity, the writer begs to present some reflections on sociology, commercialism, and the theory of engineering expenditures.

Social Structure.

As a foundation for the ideal conduct of life, there has been formulated the proposition that the greatest good possible should be striven for (whether this eliminates individual, genus, or species); that the common weal is the proper goal; and that, not only one generation or age is to be considered, but that all future generations and ages should have a proper weight in the final sum total of welfare. That is to say, all life has its standing in court, be it plant or animal, present or future.

To the end that man should have his fair share of this world's goods, there were organized the primitive protective leagues which have developed into our modern governments, one of the highest functions of which is modern communistic protection, the exercise of police power, defined as:

"That power which inheres in legislation to make and enforce all manner of reasonable regulations and laws to preserve the peace, order, and safety of society, and to prescribe the mode and manner in which every one may so use and enjoy that which is his own as not to preclude a corresponding use and enjoyment of their own by others."

To-day, man no longer needs protection against his original savage enemies. They are conquered, and he now turns his attention to other pursuits, foremost and basic of which is the production and distribution of more economic necessities and luxuries, the direction of "the great sources of power in Nature for the use and convenience of man." As mutual protection undoubtedly influenced the form of early government, we have, to-day, inevitably, a business form of government. Business is the chief occupation of the average individual. We live in a commercial age. Business and government to-day are inseparable, but, as originally, human welfare (and not government or business) is the *causa causans* of government and business.

This point deserves emphasis. The good of the people is the primary object of government and business, "business" prosperity being a sec-

ondary and incidental end, or, more correctly, simply a means to a richer and fuller life. The long-time security of investments, ultimate stability of government, and eventual greatest common weal of man are inextricably interdependent. Whatever truly furthers the one furthers all, and whatever threatens one does, in the long run, threaten all.

As the creature and creator, the servant and master, of this triple alliance (that of people, government, and business) stands the engineer. An abhorrer of greed, a seeker of the truth, a designer, builder, and operator of public utilities, he is at once: the people's protector, for the innocent investor can know at first hand nothing as to the security of his savings; the business leader, for a going concern is an engineering product; the advisor to governments, for business rightly has a voice in government, and many governments now embark in business; and the mediator in disputes arising between people, business, and government.

The actual state of affairs is a long way from that just outlined as the one to be striven for, albeit the general trend is toward, rather than away from, perfection: though not as yet, but some day, the lion will lie down with the lamb.

People collectively and individually contribute to imperfections; lacking knowledge of what constitutes the supreme good, man has builded on the philosophy of the good of man. Individually, we in all cases (to a greater or lesser degree) place the good of self first; one's self is paramount to one's fellows; altruism, the spirit of service, comes second. Collectively, we at times make unwarranted expenditures of money; we dissipate the property of posterity with a spirit of absolute indifference to the future; we war among ourselves, one institution against another.

Government, lacking the one-time incentive to the protection of the governed from external enemies, has given a part of its energy to protection from internal enemies. We now have vested interests and special privileges. Our economic ills of to-day, such as high tariff, monopoly caprice, concentration of wealth—and so on down a painful and familiar list—are laid at the door of government, apparently without realization that in America the People is the Government. Individually, we need to have nationally, a more awakened spirit, so that our business form of government will the sooner be

purged of its ills through the application of a governmental and commercial unified policy favorable to all interests.

Business, blinded by the dust of extensiveness, has at times failed to perceive its primary fundamental object, and has been guided by its secondary fundamental object: the welfare of the people has been incidental to the apparent welfare of business. With the cessation of territorial annexation and colonization is coming the more clear-sighted and deliberate period of intensiveness, so that already the short-time balance of accounts gives way to the long-time reckoning: corporations are, happily, changing their attitude—they are beginning to see farther than their noses, and consequently are considering the effect their policies have on the ultimate stability of business through their effects on virility.

Engineering Expenditures.

As engineering enters so intimately into every phase of human life, and as every engineering proposition must needs be financed, the writer is much interested in the general subject of engineering expenditures. He realizes that:

1.—In the work-a-day world, the philosophy of the good of all life is entirely too abstract and general—it is a sort of sacred standard. There is needed a more concrete and particular criterion—a field tape, as it were—in determining whether an object be worthy, which test must necessarily be, "Will it pay? How much money will it make?" The sum total of all our investments, in order that we may not deplete what capital has been handed down to us, must net at least zero; and, if we are to bequeath as much property *per capita* as we received, this sum total must net a positive quantity.

2.—By happy combinations of capital and labor we are enabled to add to the world's wealth, its increase being measured by the profitableness of the venture which depends on the acumen displayed in selecting a field, erecting a plant, and managing the business as a whole. Each of these three phases (selection, construction, operation) is becoming, through our division of labor, with its consequent concentration and specialization of effort, multiplication of possibilities, and increase in size of units, more and more difficult of proper solution.

3.—The successful engineer is the one who grasps the principle that engineering is business—that good engineering is good business—and solves the questions, "How much?" and "Will it pay?" with capable engineering business judgment. In any engineering business possibility, the engineer, before passing on the security of an investment, must know the balance between gross annual expenditures and gross annual receipts; the difference giving the net annual income, positive or negative as the case may be, but always positive except when people are in business for their health or amusement. This rigid requirement, that an undertaking must be commercially feasible, strangles the industrial application of many scientific possibilities.

Notwithstanding that an analysis of profit, to be complete, must comprise an enumeration of the factors of gross income on the one hand and of gross expenditures on the other; and, further, notwithstanding that receipts and expenses are generally interdependent, this paper deals but little with the former. It is hoped that some one will present a paper on the elements entering into gross income.

In brief, total income is the product of unit prices and the number of units sold, both fluctuating according to the law of supply and demand, or the law of monopoly distribution—in any event according to the law of variation of returns.

The factors of total annual cost—to deal with this side at somewhat greater length—are interest on first cost (unmistakably including development cost); depreciation or sinking fund; the creation of a reserve for extensions, betterments, and unforeseen contingencies; outlays for raw material and supplies; maintenance and repairs; wages, salaries, and royalties; rent, taxes, and insurance. These factors, intricate within themselves, are, in addition, involved in a complex interdependency. To illustrate:

In designing an important bridge, considerable attention may be given to the question of economic dimensions, first cost alone being considered. It is then to be remembered that first cost and maintenance are bound together, indifferent material and workmanship necessitating heavy up-keep charges. Moreover, precisely how fast depreciation should be allowed to take place and exactly to what extent it should be checked by repairs, is only one of the multifarious issues the engineer must determine. To wander back, for a moment, to receipts, the rate of exploitation of an exhaustible resource is of fundamental

importance in its bearing on first cost: Should a plant be installed capable of using all available raw material in ten years, or a thousand? As a final example, illustrative of the influence any factor of the cost of production has on every other factor, and of the weight each has in the balance between income and outgo, we may consider the rate of exploitation of a perpetual resource; more specifically, we may ponder on the height of dam in any water-supply problem as it enters into questions of cost and useful and wasted draft.

These considerations bring us to the general question of the best size of engineering expenditures ("What is the best height of dam within the limits of height giving rise to a proper return on cost," and "What is the best rate of exploitation of a limited quantity of ore," being two examples), on which question the following theory is formulated:

Of all possible arrangements of all expenditures, that one is best which secures the maximum present worth of ultimate total incomes. This statement applies to an imaginative case dealing with the sum total of all expenditures and receipts. That is to say, it does not mean that if $10 000 000 spent on a railroad will net a final profit worth now $15 000 000, whereas $20 000 000 spent will yield a sum worth now $26 000 000, then the latter amount is the better to invest, unless the second $10 000 000 would be idle or be put in an enterprise which would ultimately return a sum the present worth of which is less than $11 000 000. To generalize this limitation: all possible opportunities for investment should be considered in the light of all the revenues they would yield, and those investments should be made which would secure the maximum total of the present worths of all incomes, more or less capital being put here or there to procure this result. The example of the dam may further clarify matters: From the point of view of the dam investment alone, we want the highest rate of interest; but it may be possible, on the one hand, to put elsewhere part of the money available for this purpose at a yet higher rate, in which event we would choose some other than the highest rate obtainable from the dam; on the other hand, a dam larger than that securing the highest rate on the dam cost might yield more returns on the surplus investment than that same money could earn elsewhere, whereupon we would choose, again, some other than the highest rate obtainable from the dam. In other words, we would put our money where it would do the most good.

Time is another element included in the general statement, as may be seen from the following example: If to exhaust a body of ore this year would net $\$0.0872x$ while to save that ore for use during the fiftieth year from now would net more than $\$x$ (interest being taken at 5%), it would be good business thus to delay, \$0.0872 being the present worth of \$1 fifty years hence. Or, again, policy would dictate delaying 50 years if then more than $\$11.4674y$ and now only $\$y$ would be produced (interest being at 5%, \$11,4674 is the value of \$1, at annual compound interest, fifty years hence). Taking this same body of ore, we could mine it all in, say, one or ten years. If (disregarding other places to put money), beginning now, we mined it in one year at a net profit of $\$z$, and by mining it during 10 years at a uniform profit we obtained an annual net profit greater than $(1.62889 \times 0.0795)\ \$z$, or approximately $0.1293z$, it would be the part of wisdom to exploit at the slower rate. (Here interest has been taken at 5% per annum, \$1.62889 is the amount of \$1, at annual compound interest, ten years from now, and \$0.0795 is the annual annuity required to accumulate \$1 in ten years.) In other words, we would spend our money when, and at the rate at which, it would do the most good.

To repeat: The ideal engineering expenditures would secure the maximum present worths of net income.

To obtain this ideal distribution of financial resources among all departments of industry would necessitate two steps. Of these the first would be a set of differential investigations (for each and every commercial enterprise), enquiring into the relation between expenses and revenue, several trial solutions being necessary to determine the results of a variation in the size of physical plant and of a postponement in initial development or subsequent improvement. These differential investigations having been made from time to time, and as new possibilities presented themselves, the results would be sent to a central clearing house where the second step, that of integration, would be taken. There the higher engineering would be performed—the financial decisions made—in the light of public policy, industrial and human welfare, and the results disclosed by the differential investigations. From a purely commercial point of view, effort would be put forth to secure such an extensity and intensity of industrial activities, by introducing changes here, allowing more money for construction or operation there, hastening exploitation in one place, and

conserving resources in another, as to secure constantly, under changing conditions, the maximum present worth of net incomes ultimately derivable from all business. The men who would thus fix the world budget should necessarily be broad and sympathetic, of absolute integrity, well schooled and experienced and possessed of such a knowledge of the inwardness of things as to realize that he who would rule must be servant of all, and to base their apportionments on the proposition that, in the end, the labor and capital interests, the human and industrial interests, the interests of people, government, and business, are one.

In the commercial world we could stub along without a realizing sense of this underlying theory of engineering and business, of engineering expenditures. Indeed, we seem in cases to be absolutely uncognizant of the relation between business and people, or of the principle of compound interest. Some concerns muzzle the ox that treads out the corn, others fail to conserve resources, and others, contrariwise, are admittedly charitable. For the most part, however, our organizations fit into the system in a fairly satisfactory way, from the point of view both of the humanist and the commercialist. The secondary fundamental of business, that a concern must make money, is automatic in its application. If a business does not pay, it fails—the result is simple and unavoidable. We must get more money out of business, as a whole, than we put in (financial efficiency, so to speak, must be apparently greater than 1), and the only way to do this is to turn labor into capital by means of existing capital. Our machinery will never be completely self-acting. We, also, must work. In striking a balance between gross annual cost and gross annual income, there is, in all organizations, considered as a whole, a positive profit, except in early or bad years.

In the case of large and old companies, this profit is at times less than it should be by reason of unknown development costs. Nor is development cost the only factor of total cost sometimes neglected, depreciation being overlooked only less frequently than it; but, all in all, any going concern pays. If too many items are neglected, one way or another, it ceases to be a going concern.

Only by chance can the best arrangement of all factors of total cost be had: Proper maintenance is dependent on first cost, length of life, cost of repairs, and cost of reproduction; operating expenses

are subject to change without notice, because of improvements in the arts or fluctuations in supply prices or wages; accident frequently plays its part in the cost of the finished product. These and many other influences render the minimizing of costs a comparative matter requiring the best of engineering business judgment.

In the daily conflict of the commercial world, there is a strong temptation to overlook the theory of maximum present worths of net income (which, it may be noted, is at the bottom of conservation), to forget those generations yet to come, and to strike out, every fellow for himself and "the devil take the hindmost," in pursuit of maximum present returns.

This is due in some instances partly to selfishness, but, regardless of motive, there are always certain circumstances limiting the perfect adjustment of engineering expenditures. One of these is our lack of information of the future: we know not what a day may bring forth. Another is the fact that some people prefer to have their property within sight than to place it elsewhere more profitably. Again, many do not wish "to carry their eggs all in one basket," even though a single one promises less trouble and more gain. The fact that capital must occasionally completely develop or leave entirely untouched property considered as a legal entity, whether or not the maximum possible development is the most economical, whether or not a part of the required capital could be more profitably placed elsewhere, is still another circumstance limiting perfect adjustment of expenditures, as is also the requirement that certain funds be handled as a financial entity (be kept intact) even though a distribution of means between two or more projects would bring better results. Lack of funds is most common of all. The foregoing classes of limitations may be considered as special cases, and are unimportant in comparison with the general prevailing incomplete knowledge of present and future conditions and possibilities. The man with only a small amount to invest cannot make large outlays in search of the best opportunity, but must "hit a head when he sees it." Even those who handle large sums can, as a rule, be informed only in certain specialized fields of business, and cannot completely cover even those; and at best the heaviest bankers can know of only a few of all the places where they can make secure investments.

This absolute impossibility of sending all information to a central

clearing house, taken with the accompanying red tape, which would tie up large amounts in investigations, and in which human frailty would certainly become entangled, has given rise to local and national concerns; to a practical working engineering business organization composed of small and large units, apparently separate and distinct, but, in effect, part of a unified whole.

Small rule-of-thumb undertakings may require no engineer whatever, but the small banks and, as a rule, individuals controlling more than very moderate sums, retain engineers for advice or to design as per instructions the physical plants necessary in the every-day local affairs—such affairs as are assumed to be of a paying nature and require only limited capital.

As we get into fields of more and more importance, the engineer occupies an increasingly conspicuous position, an example of the transition from local to national engineering being given by the case where an energetic citizen makes a water-power filing, then has a preliminary investigation of possibilities made, the result of which may justify careful and detailed study by more experienced men, which study may in turn lead large financial interests to design, construct, and operate a hydro-electric plant. So we lead up: an idea, a local engineering study, a bond issue development. The local engineer handles the small work; big jobs must be done by those who can furnish the money.

There is thus seen the absolute necessity for a money trust—for a ring of men commanding tremendous amounts of capital (it may be the accumulated individual savings of a nation) furnished with accurate information as to the probable balances between expenses and incomes of large ventures, endowed with good judgment, and broad in philosophy, which ring can direct where money shall be spent, how much shall be spent, and who shall spend it. A money trust, a central authority, is an absolute necessity.

Certain abuses with which "money" is now charged are not necessary: The producer should not be underpaid while the consumer is overcharged—he is the same man, and on him rests "prosperity." Arbitrary decisions should not be made contrary to the revelations of scientific discoveries, with the object of momentarily swelling the private purse. No just cause can be found for dealing as we do now in shorts and longs—for selling what we have not got and buying what

we never expect to own. Nor can we excuse violent and sudden fluctuations in the value of stocks and bonds, with the accompanying panic or strong market, as the manipulator may desire, while the wheels of industry turn, unheeding, precisely the same. Furthermore, gain resulting from high prices and low wages should not be capitalized. "Water" is bad business. The management of investment securities with resulting transfers of certificates of ownership, which may be called financial engineering, is a necessary incidental of modern economic conditions and an entirely different thing from bull and bear skirmishes. Such things are in fact entirely foreign to industry, to good business, to sound enterprises. Such things ought not to be.

Conclusion.

Having passed through the eras of constructing temples, of erecting personal monuments, of cutting ourselves off from intercourse with other peoples, we stand to-day in the age of international commercialism, but not of commercialism at its height. Although the pendulum seems to be swinging back, although some sense again the raising of roses instead of dollars, we have only begun to see the vast field of engineering business. Commercialism, big business, is here to stay.

The application of the theory of engineering expenditures is possible only when bankers have a broad vision and a wide range of choice; they can invest money most profitably only when they know all possibilities. Economy of production and satisfaction of service are best obtained by large units, by monopolies. Competition, with its duplication of expenses, is uneconomical, and contrary to the principle of division of labor. We want one butcher, one baker, one candlestick maker. The best kind of a business is a well-managed big business. The day of little things has passed.

As each individual now has a voice in local and National government, so too each will come in time to have an actual direct share in small and in big business. Each will have some daily occupation (something to keep him busy with his immediate physical surroundings), together with securities (bonds, or shares of stock) issued by a large business concern. We will come in time to a common ownership of big business, over and above general possession of only personal effects, either by government ownership or by widespread indi-

vidual ownership of small amounts of stocks and bonds. In either event, because of the interdependency of business, government, and people, we will come—are coming—by means of enlightened self-interest, to a closer alliance between them. Enlightened self-interest is not anti-commercialism, it is pro-commercialism. The making of money is not incompatible with the making of men. Both processes will continue. Corporation publicity and the spirit of individual service are only the first feeble gasps of a new-born co-operation of business, government, and people.

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PAPERS AND DISCUSSIONS

THE SEWICKLEY CANTILEVER BRIDGE OVER THE OHIO RIVER.

Discussion.*

By A. W. Buel, M. Am. Soc. C. E.†

A. W. Buel, M. Am. Soc. C. E. (by letter).—The truss members of the anchor and cantilever arms and the suspended span of this bridge are pin-connected throughout, except where compression diagonals connect on forks forming common pin-plates at L_8, L_{10}, L_{12}, L_{18}, and M_{23}, and the connections of sub-verticals to the lower chords. The lower chords, from L_0 to L_{22}, and the top chords of the suspended span, are faced for square bearing under full load and spliced in the usual manner. Excepting the tower posts, L_{10} — U_{10}, which have 34-in. web plates, the maximum depth of these members is only 28 in. With these conditions, and reasonable care in detailing, no very large secondary stresses were found, and were not to be expected. Mr. Buel.

The 14-in. pin at L_{10} relieves the secondary stresses at that point in a manner fully as efficient as the segmental rocker bearing of the Beaver Bridge.

It is important to note that the weight and loads of the Beaver Bridge are about three times as great as in the case of the Sewickley Bridge; also, that the depths of main members of the Beaver Bridge are approximately twice as large as those of corresponding members of the Sewickley Bridge, and that similar deformations produce secondary stresses about in proportion to the depths. The same comparison holds substantially for the old Quebec Bridge.

In investigating the secondary stresses in the Sewickley Bridge, some important compensations developed, which made it unnecessary to increase the sections, except in a few instances. For example, take

* Continued from February, 1913, *Proceedings*.

† Author's closure.

Mr. Buel. the secondary stress in the extreme fibers of lower chords, L_9, L_{10}, L_{11}, due to deflection of the trusses. With maximum live load on the bridge, the additional compression in the chords of the lee truss due to wind brings the stringers into action, relieving the chord. Another point that should be considered is the difference between the character of loading for railway and highway bridges, particularly as to the probabilities of maximum live load occurring simultaneously with maximum wind load.

The secondary stresses for all the points just mentioned, and for others, were investigated, and, considering the liberal provisions made for wind stresses, the comparatively small secondary stresses were sufficiently provided for, in the opinion of all the engineers connected with the work.

Regarding the eccentric flanged bushing in the pin-holes of the shoes at L_0, on the anchor piers, introduced to provide for adjustment, a very little study of the description and illustrations should be sufficient to show that no eccentricity at all is caused by this in the anchor bars. It is true that a maximum of $\frac{1}{2}$ in. eccentricity may exist in the bearing of the shoes on the masonry, but, as the base plates are 69 by 54 in., and as the maximum possible reaction produces less than half the permissible load per square inch on the masonry, the effect of this eccentricity is negligible.

It is not strictly true that the stress in the anchor bars is constant after adjustment is made. It will be relieved by the deformations of the masonry and of the shoe with its pin-holes and pin. Although this can hardly be computed with precision, an approximate estimate indicates that the variation would be about 20% from the maximum.

Answering Mr. Hudson's question as to the cost of making the anchor arms self-supporting:

The cost of this provision may be taken, approximately, as that of the following items:

Erection ties:

$U_2 - M_3$. Two 8 by $\frac{9}{16}$-in. eye-bars.
$U_4 - M_5$. One $8\frac{1}{2}$ by $\frac{5}{16}$-in. plate.
Four 3 by $\frac{5}{16}$-in. angles,
plus details, about 25 per cent.

Reinforcing, for erection stress:

$M_3 - L_4$. Two webs changed from 15 by $\frac{5}{16}$ to 15 by $\frac{1}{2}$ in.
Four angles changed from 3 by 3 by $\frac{5}{16}$ to 3 by 3 by $\frac{3}{8}$ in.
Pin-plates and areas back of and through pin-holes reinforced.
$M_5 - L_6$. Pin-plates and areas around pin-holes reinforced.

The temporary ties, $U_2 - M_3$ and $U_4 - M_5$, were only provided for one anchor arm, as they were removed from the north side before the south side was erected. Mr. Buel.

It will be seen that, in this case, the cost of making the anchor arms self-supporting was not great. Besides the sense of security that it gave to all concerned—by no means of negligible value—it left the Erection Department more latitude in arranging the sequence of operations, thus making for economy in the field.

Although it did not become necessary to swing the anchor arms on account of high water and ice, there were two or three occasions when the hazard would have been very great had it not been for this provision. Moreover, as a matter of convenience, they were swung and the falsework removed before the stresses were reversed.

Mr. J. G. Chalfant, County Engineer of Allegheny County, has furnished the following data on the floor construction, and his experience with a similar floor which has been in service several years:

"The buckle-plates are ⅜ in. thick, from 18 to 34 ft. long, with several buckles in each, and are supported and riveted at the sides and ends, with the buckles turned down. The width of flat plate, between buckles, varies from 3 to 8 in., with a width of 11 in. in four plates. In 95% of the cases this width is not more than 6 in. The minimum thickness of the concrete is 3 in.

"The efficiency of this detail cannot be determined by calculation, and conclusions must be based on experience.

"Bridge No. 1, Allegheny River, at Oakmont, has substantially the same detail, and, after three years of service, shows no sign of failure."

The problem of making the connections in the field was given very careful study, the conclusions of which were entirely justified by the actual results.

Before adopting the lengths of truss members computed for camber, as described in the paper, a Williot diagram for cambered position was constructed, the vertical displacements being checked by computed deflections. From this a careful analysis was made of the erection conditions for each panel and joint, with particular reference to sub-panel members and to riveted field splices, such as those of the lower chord and main compression diagonals. The elevations of each bent of falsework and of the camber blocking were computed accurately, with allowances for settlement, etc., based on experience, as were also the positions of all lower-chord pins in relation to the traveler sills and rails. These elevations, with the clearances allowed for, were shown on erection plans. After this work was completed and checked, the engineers of the Fort Pitt Bridge Works were satisfied that no unusual difficulty would be experienced in the field.

Mr. Buel.

The following quotations from the erection reports give the recorded results:

"As a rule, and speaking generally, in all riveted connections about 90% of the holes were filled with temporary bolts and drift-pins. Drift-pins were used in the field connections of members taking tension during erection, as bolts would not take up the clearance allowed in the holes, but all riveted members were connected practically without the use of drift-pins.

"All pins were driven home quickly and without the slightest distortion or displacement of metal about the holes. Special rams, weighing about 2 000 lb., were sent to the site, but were never used, as it was found that all pins could be driven easily with a 15-ft. section of 85-lb. rail."

Although it is recognized that the problem of providing for the field connections in bridges of such proportions as the Beaver, Blackwells Island, or Quebec is quite a different matter from that met in the Sewickley Bridge, it may be worth while to consider the causes of the rather unusual facility with which the latter went together in the field, even when compared with structures in its own class. There is no doubt that the shop methods, as mentioned in the discussion by Mr. Straub, had considerable to do with it, and the form and type of trusses and details contributed to the results; but the methods used for introducing the camber and for checking its effect on field connections, together with the care taken to set the falsework and camber blocking to correct elevations, seem to be the most important items in accounting for the exceptionally satisfactory results attained. The accuracy of all these computations, as well as of the shop work, is attested by the remarkably close agreements between the computed and actual deflections, as shown on Plate LXXXVII.*

* *Proceedings*, Am. Soc. C. E., for September, 1912.

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TUFA CEMENT, AS MANUFACTURED AND USED ON THE LOS ANGELES AQUEDUCT.

Discussion.*

By J. B. Lippincott, M. Am. Soc. C. E.†

J. B. Lippincott, M. Am. Soc. C. E. (by letter).—The writer is gratified at the generous number of contributions that have been presented in the discussion of this paper, especially as arguments have been given vigorously on both sides of the question. Before entering into a review of the discussion, it is desirable to add certain data which have become available since the paper was written. Mr. Lippincott.

On page 1199‡ there is given a test (up to an age of 6 months) of a mortar made of 75% of hydrated lime and 25% of tufa. The 1-year break shows a strength of 252 lb., an increase of 27 lb. over the 6-months break. This briquette shows a continued hardening in water, and demonstrates the hydraulic properties of the tufa in combination with lime.

Table 4 gives the strengths of Haiwee and Fairmont tufa cements up to an age of 1 year. Unfortunately, briquettes for longer time tests were not kept during the early stages of the work on the Los Angeles Aqueduct. A series is now being held for this purpose, but the 2-year period has not yet been completed. For briquettes made of tufa cement consisting of 50% of tufa and 50% of Monolith cement, mixed with 3 parts of sand, four breaks, made at the Haiwee mill, indicate a growth in strength from 500 lb. at the end of 1 year to 595 lb. at the end of 1 year and 289 days. Breaks of the oldest briquettes have just been made at the Fairmont plant, with the following results, each figure representing an average of ten tests. The cement used was

* Continued from March, 1913, *Proceedings*.

† Author's closure.

‡ *Proceedings*, Am. Soc. C. E., for October, 1912.

Mr. Lippincott. 50% of Monolith and 50% of Fairmont tufa mixed with 3 parts of standard sand:

3 days.	7 days.	28 days.	3 months.	6 months.	1 year.	1 year, 319 days.
153 lb.	240 lb.	388 lb.	467 lb.	492 lb.	555 lb.	565 lb.

More breaks were not made because it is desirable to preserve the series for testing at regular periods. They indicate the continued hardening of the tufa mortar with age, which is characteristic.

Table 5 shows tufa sand briquette tests with varying proportions of tufa for ages up to 6 months. Table 17 gives results in continuation of that table.

TABLE 17.—Tufa Sand Briquette Tests, with Varying Proportions of Tufa: A Continuation of Table 5.

Briquette number.	Percentage of tufa.	Tensile Strength.					
		3 days.	7 days.	28 days.	3 months.	6 months.	1 year.
21	55	85 90	200 210	335 345	370 370	330 395	410 400
22	60	30 35	100 100	300 310	460 435	390 385	520 460
23	70	35 35	90 100	250 260	350 355	330 365	460 430
24	75	15 25	75 80	210 200	300 285	325 375	370 270
25	80	15 20	75 80	150 160	255 235	300 305	415 395
26	80	15 20	70 70	90 90	140 110	230 260	310 265

The cement used (in the tests in Tables 5 and 17) was Monolith, and the tufa was from the Monolith quarries. A good increase in strength is shown between the 6-months and the 1-year breaks. Especial attention is called to the strengths shown in the 1-year breaks of the 80 and 85% tufa cements.

On page 1213* is recorded the results of tests with a blend of Monolith cement and diatomaceous earth in the proportions of 50% of each. It should have been stated that this blended cement was mixed with 3 parts of sand.

Table 10 states the results of tests made with a mixture of volcanic clinkers and lavas found on the Hawaiian Islands with Santa Cruz (California) cement in equal parts by volume. Few tests were available for longer periods than 28 days at the time of writing the paper. Table 18 contains the results of tests for periods up to 1 year.

* *Proceedings*, Am. Soc. C. E., for October, 1912.

Mr. Lippincott

TABLE 18.—SAND BRIQUETTE TESTS MADE WITH A MIXTURE OF HAWAIIAN LAVA AND SANTA CRUZ CEMENT: A CONTINUATION OF TABLE 10.

Number of tests.	Date made.	Percentage water.	Tensile Strength. 3 days.	7 days.	28 days.	3 months.	6 months.	1 year.	Brand.	Percentage of lava.	Fineness. 100 M.	200 M.	Setting Time. Initial. H.	Initial. M.	Final. H.	Final. M.	Boiling test. 6 hours
2	1911. Nov.	10½	140	255	340	450	465	480	....	0	93.8	....	2	15	6	30	O. K.
18	Dec.	10½	112	190	307	400	467	470	Red Clinker.	50	....	92	2	30	6	00	"
10	1912. Jan.	10½	168	148	230	295	388	391	Lava.	50	....	92	1	..	11	..	"

The tests in Table 18 show fair results, and indicate a cement suitable for ordinary hydraulic work. This cement is not as strong as the blends made with the tufas already described. The tests show a uniform growth in strength with age.

Some tufa cement was sent to Robert E. Ford, Professor of Mechanical Engineering of Throop Polytechnic Institute, at Pasadena, Cal., with a request that it be tested for compression. Standard briquettes were made and first broken in tension and the two halves then crushed. Cubes, 2 in. square, were also crushed, and the ratio of tension to compression was obtained. An unusual quantity of water was used by Professor Ford to get normal consistency (28½% for neat cement), and this, possibly, may account for the fact that the values are lower than those usually obtained in the city laboratories. The cement was Monolith, and the tufa was from Fairmont. Standard sand was used. Table 19 is a summary of the tests. The results for compression compare favorably with tests given by Taylor* for Portland cements, but these cements are not as strong as the German trass cements (the tests of which are given in Table 9), nor are they as strong as those mentioned in Table 13, of materials tested at the Arrowrock Dam, as quoted by Mr. Paul, either for their blended material or for their straight cement mortars.

Mr. O'Hara states that Professor Eakle, of the University of California, considers that "the rock ground with the cement used on the Los Angeles Aqueduct, is a rhyolite-tuff and a trachyte-tuff, which, when finely ground, will possess the same characteristics as finely-ground clay."

Mr. O'Hara continues:

"The cement produced by the method used at Monolith and Haiwee, Cal., is similar to cement adulterated with clay. Rhyolite-tuff and

* "Practical Cement Testing," p. 215.

Mr. Lippincott.

trachyte-tuff are not to be confounded with the volcanic rocks known as puzzuolana and trass, which have been used for the manufacture of cement. Puzzuolana and trass are the hardened products of volcanic action in their original state, in which respect they are similar to blast-furnace slag. On the other hand, the volcanic tuff of the nature found at Monolith and Haiwee, is not comparable to blast-furnace slag, being more of the nature of altered volcanic rock."

TABLE 19.—COMPRESSION TESTS OF LOS ANGELES AQUEDUCT TUFA CEMENT (MONOLITH).

Briquette No.	Cube No.	Days cured.	Ratio: Tension-Compression.	Average Tension.	Average Compression.	Average ratio: Tension-Compression.
NEAT BRIQUETTES.						
5a......	....	7	6.8	270	.....	....
....	5m	7	7.6	...	1 945	7.2
4.......	....	28	8.0	...	.....	....
5b......	....	28	8.0	...	.....	....
6a......	....	28	7.3	...	.....	....
6b......	....	28	9.3	490	.....	....
....	5n	28	8.95	...	.....	....
....	6m	28	8.7	...	4 125	8.4
5c......	....	180	11.4	...	.....	....
6c......	....	180	14.0	540	.....	....
....	4m	180	15.35	...	.....	....
....	6p	180	13.75	...	7 370	13.6
3 TO 1 STANDARD SAND.						
7a......	....	28	4.55	...	.....	....
7b......	....	28	4.3	...	.....	....
8a......	....	28	5.2	...	.....	....
8b......	....	28	5.5	265	.....	....
....	7m	28	4.0	...	.....	....
....	7n	28	4.1	...	.....	....
....	8m	28	4.6	...	.....	....
....	8n	28	4.65	...	1 220	4.61
7c......	....	180	6.25	...	.....	....
8c......	....	180	5.95	...	.....	....
8d......	....	180	5.5	367	.....	....
....	7p	180	5.8	...	.....	....
....	8p	180	6.9	...	1 225	6.1

Table 2 gives an exact chemical analysis of the tufas used on the Los Angeles Aqueduct, of Italian tufa, and also of Italian puzzuolana (as used in that country), as determined from samples furnished by the American Consul. A detailed analysis of the German trass is given in Table 20 where it is shown that it has a close resemblance to the California tufas. The local clays analyze much more closely to the Italian puzzuolanas than the California tufas. The percentage of silica in the local tufas, soluble in alkalies, is greater than that in the samples of Italian puzzuolanas, in the ratio of 4 to 1, according to determinations by the Monolith Laboratory, and this is believed

to be a very important factor. Mr. O'Hara is mistaken when he refers to these tufas as an altered volcanic rock. If he had ever seen these quarries, he probably would not think so. The tufa used is an unaltered volcanic ash which has been thrown down in water and comminuted. It has been partly consolidated by pressure. Physically, it has no more resemblance to clay than unground cement clinker, which also contains chemically a high percentage of clay. It resembles somewhat the pumice stone which can be purchased at most drug stores. Many of the hardest rocks, such as granite, slate, and lava, on decomposition form clay, and it would be as misleading to say that a granite is a clay as to say that this tufa is a clay. The writer is familiar with tufa in its altered and clay-like condition. It is found near Haiwee. Mr. Lippincott.

The tufas are very low in sulphur, containing usually less than one-tenth of 1%, and it is difficult to see how there is any extra danger because of being blended with straight cements containing 1½% of sulphur, on this account, as Mr. O'Hara states. Reference is made to the discussion by Mr. Luiggi as to the enduring quality of these cements in sea water.

Mr. O'Hara also says:

"Tufa and Portland cement, as used on the Los Angeles Aqueduct, is a mechanical mixture, the two materials being blended in equal parts by volume. Under these conditions, no silicates of lime are found, and the gradual increase in strength is not due to the same cause as the slow hardening of a high silica Portland cement."

The chemical analyses of the Bureau of Standards, given on page 1206,* as well as the analyses at the laboratories of the Los Angeles Aqueduct, show that silicates of lime are formed between the tufa and cement. Mr. O'Hara's statement, as given, is only empirical, and should not outweigh these chemical determinations. The writer regrets that Mr. O'Hara considers the practice of blending tufa cements on the Los Angeles Aqueduct as "dangerous and without precedent." Many unprecedented things are done, especially by the Engineering Profession, which result in general benefit. There is comfort in the fact that the U. S. Reclamation Service is now building or operating three cement blending and re-grinding plants, after careful investigations, made by its engineers, as shown by the instructive discussions contributed by Messrs. Rapier R. Coghlan and Charles H. Paul.

Mr. Reed calls attention to the severe climatic conditions under which the concrete work on the Los Angeles Aqueduct is carried on. The relative humidity in the Mojave Desert during the summer is about 20%, and this, together with temperatures which are often greater than 100°, and the great scarcity of water, makes a most try-

* *Proceedings*, Am. Soc. C. E., for October, 1912.

Mr. Lippincott. ing condition for concrete work. Except on the 40 miles of open canal described by Mr. Peterson, methods of work were devised which kept the green concrete shielded from the sun and wind. Standing water is usually maintained in the conduit with small dams, to give an artificial humidity. Doors or curtains were maintained at tunnel portals. The roof was cast on the outside conduit before the forms were taken from the sides, and it was covered with wet earth on top as soon as the concrete was poured. The bottom was placed last, and the manholes were kept closed. It was only by such methods that shrinkage cracks could be avoided, no matter what kind of cement was used. In this way, mile after mile of the outside conduit could be built without any expansion joints and without transverse cracks. Leaving a manhole open for a week or two would often cause a transverse crack to open within 10 ft. Where the conduit has been filled since, these cracks, in many instances have swelled shut.

Mr. Richardson states that:

"Unfortunately, no data are available showing what proportion of the silica in any of the materials occurring abroad, or in those in California, is soluble in acid, as it is hydrated silica of this description which possesses the greatest hydraulic value."

It is the writer's understanding that it is solubility in an alkali solution which is desired. This laboratory determination is made with a dilute solution of sodium carbonate. On page 1206* the U. S. Bureau of Standards is quoted as showing that 7.17 out of 30.97 parts of the silica in the briquettes made of Fairmont tufa cement, or 23%, were soluble in the mortar. A test of the pure pulverized Monolith tufa, in the Monolith Laboratory, showed 8.4% soluble.

Comparative analyses of the German cements and the Monolith tufa are given in Table 20. The information in reference to the German materials (referred to in Table 9) is from the report of the Royal Testing Laboratory.†

TABLE 20.—ANALYSES OF GERMAN TRASS AND MONOLITH TUFA.

	German trass.		Monolith tufa.
Loss on ignition	3.29	10.25	6.28
Water of crystallization	6.96		
Insoluble silica	41.56	68.49	69.48
" "	1.39		
Soluble silica	25.54		
Iron oxide	2.27		2.52
Aluminum	9.83		11.37
Lime	1.05		1.80
Magnesia	0.67		2.95
Potassium	3.04		Not determined.
Sodium	3.09		" "
Phosphorus	0.11		" "
Sulphuric acid	0.06		0.43
(Unable to translate)	0.10		....
	98.96		

* *Proceedings*, Am. Soc. C. E., for October, 1912.

† Mitteilungen aus dem Königlichen Materialprüfungsamt, 27 Vol., 1909, p. 348.

These analyses show a striking similarity between the German trass and the Monolith tufa. The quantity of sulphur shown in Monolith tufa is unusually large in this sample, and the 69.48% is for total silica, no division having been made between soluble and insoluble. Mr. Lippincott.

The sample of Italian tufa available at the Monolith Laboratory shows only 2% of silica soluble in dilute sodium carbonate. This sample was furnished through the American Consulate, and possibly may not be typical. What was furnished as Italian puzzuolana shows practically the same.

Replying further to Mr. Richardson, the tufa is separately ground in the ball mill, so as to pass through a 20-mesh screen, but the final grinding of the blended tufa and cement is done in the tube mill. An effort was made to grind the cement clinker with the crushed tufa, but it ground very slowly in the tube mill. As it was also desirable to save transportation charges, the final grinding and blending were done mostly at distant points on the work, where tufa was available, thus saving half of the freight.

Mr. Wagoner quotes from a printed "Report on Municipally Manufactured Cements of the Los Angeles Aqueduct" made to the American Portland Cement Manufacturers Association, by a party of five engineers. The circulation of the report has been suspended by the Association. As an example of its bias, an illustration is given of a test slab which was not a portion of the Aqueduct, and had been loaded to destruction by the city's engineers, in order to test the concrete materials, as used in construction. In the pamphlet this is labeled "Slabs of poor concrete, where cover on L. A. Aqueduct has already collapsed and been replaced, on desert west of Mojave." The writers of that report state that there are three kinds of concrete on the Aqueduct: good concrete made of purchased cement; fair concrete made of Monolith cement; and poor concrete made of tufa cements. They state the places where these concretes are found, and it is amusing to note that their "poor concretes" in most cases actually were made of purchased cements.

The Monolith mill has been run for four years. The product, with minor exceptions, has been standard. The hydraulic index which Mr. Wagoner has taken exception to, has usually been about 2.0. It is not claimed that the mill is infallible. Neither are other mills. It has been watched closely by cement manufacturers, and when it was in temporary trouble, due to a change in the limestone quarry, it was promptly investigated. The writer regrets that it is necessary to refer at all to this report. Much more could be said, but it would involve local politics and business foreign to this discussion. Table 21 contains an independent analysis of Monolith cement.

Mr. Wagoner suggests that tests with tufa and some standard cement be made for the Profession. Many barrels of tufa cement have

Mr. Lippincott.

TABLE 21.—COMPARISONS OF STANDARD CEMENT SPECIFICATIONS WITH TESTS OF MONOLITH (LOS ANGELES CITY) CEMENT, FURNISHED BY THE PARK DEPARTMENT, AS TESTED BY SMITH EMERY COMPANY, FEBRUARY, 1913.

PHYSICAL TESTS.

Authority.	Tensile Strength, in Pounds per Square Inch.				
	Neat, Pure.			1 part cement and 3 parts sand.	
	1 day.	7 days.	28 days.	7 days.	28 days.
American Society for Testing Materials. Specifications	175	500	600	200	275
U. S. Bureau of Standards. Specifications	None	500	600	200	275
Smith Emery Company Test	182	632	798	202	273
	216	709	800	214	285
	244	656	838	215	280

Time for 28-day test not yet expired.

CHEMICAL TESTS.

	U. S. Bureau of Standards. Specifications. Percentage.	American Society for Testing Materials.	Tests by Smith Emery Company.		
			No. 1.	No. 2.	No. 3.
Silica	19 to 25	....	25.02	24.72	24.80
Alumina	5 to 9	....	5.58	5.40	5.52
Iron oxide	2 to 4	....	2.72	2.00	2.48
Lime	60 to 64	....	62.70	63.20	62.92
Magnesia	1 to 4	Not more than 4	1.76	1.98	1.66
Sulphur trioxide	1 to 1.75	Not more than 1.75	1.14	0.95	1.08
Loss on ignition	0.5 to 3.0	....	1.80	2.00	2.00
Insoluble residue	0.1 to 1.0	....	....	....	....
Fineness, through 200-mesh sieve	75% or more	75% or more	81.2	80.4	80.8
Fineness, through 100-mesh sieve	92% or more	92% or more	94.0	94.0	94.2
Setting time	Not less than 45 min.	Not less than 30 min.	3 hours.	3 hours.	3 hours.
Boiling test	Boil without cracking.	Boil without cracking.	O.K.	O.K.	O.K.
Specific gravity	Not less than 3.10.	Not less than 3.10.	3.12	3.12	3.12

The ratio of the weight of the aluminum silicate to that of the lime is called the Hydraulic Index, and should be as high as 1.91, the German cements showing 1.98. This analysis indicates for Monolith cement 2.08. (Note by J. B. LIPPINCOTT.)

been made with both Colton and Riverside cement, and they show the same results as those made with Monolith cement. Mr. Lippincott.

In reply to Mr. Mogensen, it may be stated that no discriminations have been made against tufa cements. Two reinforced concrete towers have been successfully built of tufa cement, as well as several flumes, and nine reinforced concrete pipes, 10 ft. in diameter, for heads up to 80 ft. Many miles of reinforced roof slab, 8 in. thick, with a span of 11 ft. 5½ in., have been successfully laid. The Haiwee Dam referred to is an earthen structure with a clay core; no concrete was used in its construction. Numerous prominent engineers have examined the work, and, although most of it is in a remote and unoccupied region, every effort has been made to aid investigations by the Profession.

Approximately, the following quantities of cement of various classes have been used to date on the Los Angeles Aqueduct:

Monolith standard cement..................	900 000 bbl.
Tufa cements..............................	636 000 "
Purchased standard cements................	260 000 "

The contributions to the discussion by Messrs. Coghlan and Paul are of particular interest and value. Little may be added to these discussions; they are commended to those interested in this subject. The writer agrees with Mr. Coghlan's statement, referring to materials suitable for blending, "that they are all dependent for their fitness on the quantity of colloidal or active silica"; and that they should "be in an almost unaltered condition." Mr. Paul's tests, showing the results of blending the granites at the Arrowrock Dam of the U. S. Reclamation Service, are especially interesting, as they broaden the opportunity for this economy. The writer's experience with the lavas from the Hawaiian Islands, though very limited, is in accord with that of Mr. Coghlan.

It must be constantly borne in mind by Eastern engineers, in considering this subject, that the prevailing price of cement at central points in the Far West, is about $2 per bbl., and that transportation charges are high. Engineers in this region are anxious to build the necessary works with the available funds. The saving on the Los Angeles Aqueduct, due to the use of tufa cement, will amount to about $700 000.

Mr. Peterson's discussion gives the experience of an engineer who has used tufa cement for three years, and, for this reason, it is of value.

Mr. Luiggi, who is Inspector General of the Royal Civil Engineers (of Italy) and Professor of Maritime Construction at the Royal School of Civil Engineers, gives a most interesting discussion of the subject.

Mr. Lippincott. referring both to the old Roman cements, made of lime and puzzuolana (tufa), and the blending of Portland cement with puzzuolana. He has practiced this latter blending since 1888. He approves the practice, and states that it is "quite correct, both with regard to the cost and the durability of the work."

Mr. O'Hara quotes Professor Eakle, of the University of California, as stating:

"The rock ground with the cement used on the Los Angeles Aqueduct, is a rhyolite-tuff and a trachyte-tuff, which, when finely ground, will possess the same characteristics as finely ground clay."

Mr. O'Hara then makes the deduction that "the cement produced by the method used at Monolith and Haiwee, Cal., is similar to cement adulterated with clay." Presumably, he gets this information from the pamphlet issued by the Association of American Portland Cement Manufacturers. The writer, having communicated with Professor Eakle and called his attention to the paper and to Mr. O'Hara's discussion, received, under date of February 27th, 1913, the following:

"Your letter of February 26th, informing me that I had been quoted by cement manufacturers was news to me, and so I have looked up the quotation to which you refer. I am rather in the dark regarding the matter, as both the gentlemen using my name and the rock in question are unknown to me. However, I constantly receive samples of rock for identification, and always try to accommodate the sender with whatever information I can give, and as to the persons sending me material seldom give me the localities or state what they wish to do with my report, I only hear of it later by some roundabout way as this."

After receiving this communication, the writer forwarded samples of the tufa from the Fairmont quarries to Professor Eakle for indentification and for an expression of opinion. He was requested to put this in such form as to permit of publication. The following letter has been received in compliance with this request:

"The samples of tuff you sent me cannot be classed as strictly fresh rocks, as the original more or less glassy base has become devitrified, a portion of the alkalies leached out, and the tuff is considerably hydrated. The alteration has, however, been more in the nature of an incipient kaolinization and silification producing a hardened stony base, as shown by the gritty powder, so the value of the rock for the use you make of it has not materially suffered by the slight change. The amount of alumina in the tuffs is rather high and there is no doubt that an extra amount of clay-like material is added to the cement, but your experiments and those of German cement users seem to prove conclusively that such rocks do not act as adulterants, but, on the other hand, produce a better and stronger cement, if properly used. Much of the free silica and alumina silicate of these tuffs is in a more soluble state than it is in ordinary clay and quartz sand, and it is presumably due to this fact that the tuffs have their value. Their

solubility is shown by treatment with HCl and with alkali carbonates, and is also shown in the field where many of them have become silicified and often impregnated with opal, deposited from the colloidal silica formed. When these tuffs are ground so intimately with cement, every portion of this soluble silica and silicate comes into contact with the lime, and during the gradual drying out of the moist mass, a chemical action is taking place to form lime silicate. The longer you can keep the interior mass moist, the more complete should be the crystallization and the stronger the mass. In quick setting we would expect the tuffs to act more as an adulterant. It would be interesting and of value to know if thin sections of the cement at different periods of solidification would not show what is taking place. The tuffs you sent are rhyolite tuffs. Your article states you use equal parts of cement and tuffs, but the analysis of the Monolith tuff cement shows a proportion of about one part tuff to three parts cement. There is a discrepancy in these analyses. The tuff cement has more water than the tuff and the amount of alumina is not larger than some Portland cements contain, and the total allows nothing for alkalies which must be present. Mr. Lippincott

"I would like to offer a suggestion about the name 'Tufa Cement' which you use. It has become the custom with petrologists and geologists, at least in this country, to use the word 'tufa' for calcareous deposits, and 'tuff' for volcanic ash rocks; so it might be better to call the cement a 'tuff-cement.'

"You are perfectly welcome to make use of anything in this letter you may find of assistance."

With reference to Professor Eakle's comment on a discrepancy in the analyses, it should be remembered that the pulverized tufa is blended with the cement at the mills in equal parts by volume. The ratios are: 60% by weight of cement to 40% by weight of tufa, approximately. The mixture for this sample, the analysis of which is given in Table 2, was of equal parts by volume. The analyses, of course, show percentages by weight. According to the writer's computations, there is the following relation between the analyses given and the theoretical quantities which should occur:

	Silica.	Iron and alumina.	Lime.
Theoretical quantities....	41.1	10.87	38.19
Analysis	35.34	11.89	41.05

The percentages shown in these different analyses, either for the cement or for the tufas, will vary from time to time, and cannot be expected to check absolutely. The quantity of silica in the tufa used in this blend is unusually high.

Mr. O'Hara also quotes from the U. S. Corps of Engineers,* as follows:

"Puzzolan cement never becomes extremely hard like Portland, but puzzolan mortars and concretes are tougher or less brittle than Port-

* Professional Paper No. 28, p. 1779.

Mr. Lippincott. land. * * * It is unfit for use when subjected to mechanical wear, attrition, or blows."

In order to test the tufa cements for attrition, twenty pieces of briquettes of tufa cement (50% by volume being tufa and 50% by volume Monolith cement, mixed with 3 parts of standard sand) which had attained an age of one year, were revolved in the small laboratory ball mill with flint pebbles; and twenty pieces of broken straight cement briquettes which had been tested for tension, which were a year old, and had been mixed with 3 parts of standard sand, were treated in a similar manner. The tensile strength of the briquettes of tufa cement was about the same as those of straight cement, each set having averaged more than 400 lb. The briquettes were weighed separately before being subjected to the rattler test, and were weighed at equal intervals of time as the test proceeded. The briquettes of tufa cement were run separately from those of straight cement, because it was difficult to distinguish between them after the grinding had proceeded for some time. The results are given in Table 22.

TABLE 22.—ATTRITION TESTS OF TUFA CEMENT AND STRAIGHT CEMENT BRIQUETTES MADE WITH 3 PARTS OF STANDARD SAND.

Time	5 minutes.	20 minutes.	50 minutes.	2 hours.	5 hours.
Straight	5.26%	10.85%	14.23%	28.007%	48.13%
Tufa	3.13%	7.66%	13.93%	28.9%	63.41%

The percentages represent the loss in weight.

This test indicates that, for periods up to 50 min., the tufa briquettes stood abrasion better than those of straight cement; that, for the 2-hour run, they were practically the same; and that on the 5-hour run the straight cement showed a superiority of about 30 per cent. It has been established in the tensile strength tests, that the tufa is slower in hardening than the straight cement, and it would appear that the outer portion of the tufa briquettes had attained a greater hardness than those of straight cement, but that the interior was not yet as hard.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

PREVENTION OF MOSQUITO BREEDING.

Discussion.*

By Spencer Miller, M. Am. Soc. C. E.†

Spencer Miller, M. Am. Soc. C. E. (by letter).—The writer is much gratified with the reception accorded his paper and the thoughtful discussion it has brought forth. Mr. Gray intimates that the writer's experience with the mosquito problem has been largely obtained in territory where mosquitoes are more of a nuisance than a menace to health. Are not all kinds of mosquitoes a menace to health? Nature provides a period of sleep to maintain health. Mosquitoes which disturb that sleep surely affect one's health; and the greater their number the greater the menace. Recreation in the open air and sunshine maintains health; therefore, a pest of mosquitoes of any variety that drives one indoors is a menace to health. Mr. Miller.

There are many times more non-malarial than malarial mosquitoes breeding in the State of New Jersey. The malarial mosquito may breed almost anywhere, even in salt marshes, and any campaigning against it must, of necessity, call for the inspection of all quiescent waters. The cost of inspection is the largest part of the cost of extermination. Seeking for malarial mosquito larvæ will lead the inspector to the larvæ of non-malarial mosquitoes. The destruction of non-malarial larvæ adds but a trifle to the expense of a campaign against malaria, and, in the interest of health, such a campaign should be directed against every kind of mosquito.

Mr. Gray states that the problem "becomes an urban problem only when there is a lax administration of the municipal departments of health and public works." In the City of New York, the Department of Health is fully alive to the importance of mosquito prevention. It

* Continued from March, 1913, *Proceedings*.

† Author's closure.

Mr. Miller. is lax only because hampered by lack of funds. A large portion of the Borough of Brooklyn is pest-ridden with salt-marsh mosquitoes which breed in about 9 000 acres of marshes in Jamaica Bay within the limits of the Borough. The Borough authorities have not $10 000 to expend for this purpose at present. It will cost about $60 000 to ditch these marshes alone, and it is estimated that an annual expenditure of about $150 000 per year will be required to relieve properly the Borough of Brooklyn from the mosquito pest. This sum would cost the people residing there less than 10 cents per annum per inhabitant. While salt-marsh mosquitoes probably furnish 75% of the pest in Brooklyn, there are thousands of fresh-water breeding places where the common house mosquitoes breed. These are a greater nuisance individually than the marsh mosquitoes, because they get indoors and sing and bite all night.

Mosquito breeding in Prospect Park, Brooklyn, has been known and talked of publicly for the last ten years, yet at the present time the City health authorities have not the funds to prevent it. Citizens of Brooklyn have formed an association and are raising a fund of $60 000 by subscription to start the work.

The writer became interested in forming county mosquito extermination commissions because, after ten years of endeavor, he failed to obtain the co-operation of adjoining communities to do anti-mosquito work effectively.

Mr. Gray concurs in the "selection of the county as a proper unit of mosquito control operations * * * with the condition that the field of the county's operations shall not include incorporated cities."

The Board of Health of Newark, N. J., for five or six years, expended about $5 000 to $7 000 annually. This was about one-sixth of the sum required to do the work effectively within the city limits. After the work was taken over by the County Commission, in the summer of 1912, more than 9 000 breeding places were discovered in the city, entirely outside of salt-marsh areas. Newark has a well-organized and highly intelligent Board of Health, but it was limited financially. Breeding was stopped to a certain extent, but not enough to satisfy the citizens, as will appear from the following editorial comments in Newark papers:

The *Newark Evening Star* on the 15th of July, 1912, said:

"In no previous summer season were Newark and its suburbs so free from the pest as during the present season. Last year in many localities the infliction was unendurable, and the change is most grateful to a large population. The money appropriated for this blessed work is the best public investment ever made by the county. It is being richly repaid in benefits."

On August 17th, the same paper stated: "The mosquito extermination act has been amply justified by results." Mr. Miller.

The *Newark News*, on July 13th, 1912, said:

"After just two months of activity, backed by an adequate appropriation, it has been demonstrated that the work of the Essex County Mosquito Extermination Commission is accomplishing definite results. * * * After this comparatively short time, results are visible in the lessened number of the pests that find their way into houses and within the confines of screened porches."

On July 24th the *Newark Evening News* stated:

"Sh-h-h!

"Have you noticed it, or are we dreaming! It seems almost too good to be true.

"Has the wily mosquito left us?"

The writer feels that it is necessary to include incorporated cities in connection with county work. A county tax is paid by the taxpayers of the cities as well as the villages. A county commission should expend the money collected by taxation for the benefit of the people who pay it.

Mr. Gray is convinced that "the appointment of special commissions is an unnecessary multiplication of executive bodies in the county government"; and states, further, that "no department of county government, on the average, is remarkable for its intelligence and efficiency"; and "that there is no assurance that a special commission will show greater efficiency than the existing component parts of the county government."

The assurance offered in the New Jersey law, included in the paper, that these commissions shall be intelligent and efficient, is that they shall be appointed by the Supreme Court Judge of each county and shall serve without salary. Mr. Gray states that, in California, there is a crying need of anti-mosquito work, and that practically nothing has been done by the county governments.

He states further: "the county officials have provided no funds * * * although the economic loss to this county [Tehama] from malaria is certainly not less than $100 000 annually." No better argument can possibly be put forth for the necessity of the appointment of specially selected men for carrying out a definite public work.

Mr. Gray says that "highly successful prosecution, requires marked abilities as an educator and publicist, as well as a unique combination of thorough knowledge as to engineering, parasitology, hygiene, vital statistics, law, and government."

The Essex County Mosquito Extermination Commission is composed of: Two doctors, one civil engineer, one retired quartermaster-general, one successful business man, and one grain merchant, all hav-

Mr. Miller. ing proven by previous service that they were public-spirited citizens. This Commission has been successful.

Mr. Rosenthal refers to the mosquito pest in the Philippines, which followed the introduction of numerous uncovered wooden water tanks and barrels, "man-made water holders."

Mr. Felt speaks with authority when he refers to the incidental pool, roadside or otherwise, as being more prolific in the production of mosquitoes than the larger bodies of water so frequently viewed with distrust by the general public. His suggestion to suspend a small container filled with oil and adjusted so as to drip slowly into a catch-basin has been tried in Newark, N. J. The plan proved effective only when the catch-basins were kept clean. After some days they become partly filled with dirt, and on this the oil may drip and not reach the water; for that reason the plan was abandoned. The catch-basin covers are now removed, and the oil is sprayed all over the surface of the water. What is wanted is a mosquito-proof catch-basin which needs no inspection or oil.

Dr. Hunt views the subject largely from a medical standpoint. The Supreme Court Judge of Essex County was fortunate in having found a man of Dr. Hunt's ability and zeal to serve as a commissioner. Where county commissions are appointed by Supreme Court Judges, the latter would do well to select one or more progressive medical men to serve on the commission, because such men can usually be depended on to be intelligent and unselfish in their desire to relieve the people from the pest.

Mr. Rutherfurd refers to the problem of Connecticut and to a Bill placing the marsh-drainage work and $200 000 in funds in the hands of the Director of the State Agricultural Station. The conditions in Connecticut are peculiar, fully 75% of the population being affected by a mosquito pest breeding along the borders of the State. There is hardly a county which will not be benefited by the elimination of the salt-marsh pest; it is a State nuisance, and a good beginning will be made by the passage of the Bill.

The writer endorses Mr. Britton's suggestion, in which he states that "a great educational campaign is needed." During the season of 1912 the Essex County Mosquito Extermination Commission issued 50 000 cards to be hung in houses. These cards illustrate how mosquitoes breed, and give methods for the prevention of breeding.

The real value of an educational campaign, however, is in moulding public opinion to uphold organized effort for anti-mosquito work. One error frequently made is to expect that people when they know how the mosquito pest can be eliminated, will look after their own premises individually. Experience during ten years has taught the writer that less than 5% of the population will do anything of the sort.

No better illustration of that can be found than the fact that the inspectors employed by the Essex County Mosquito Extermination Commission found mosquitoes breeding in the rear of the writer's stables. Mr. Miller.

In reference to the cost of anti-mosquito work, the following data may serve as a rough basis for making estimates:

1. Port Said, with 50 000 population and compactly built, controls its mosquito pest for 12 cents per inhabitant per annum. Ismailia, an open village, with 10 000 population, expends 50 cents per inhabitant per annum.

2. Klang and Port Swettenham, 5 miles apart, have a combined population of 3 000 and spend $4.72 per inhabitant.

3. Mr. Means, in his discussion, refers to anti-mosquito work in the Sacramento Valley, Cal., in a territory of about 10 000 acres and 1 200 people at $3 per inhabitant per annum. This, however, is only 36 cents per acre per annum.

4. The estimated cost for the Borough of Brooklyn (Kings County), with a population of 1 700 000, is 10 cents per inhabitant per annum.

5. In South Orange, N. J., a suburban community of 5 000 population, the cost of effective work is about 30 cents per inhabitant per annum, or an average of $1.50 per acre per annum.

6. The estimated cost of keeping New Jersey reasonably free from the pest is 29 cents per inhabitant per annum, or an average of 13 cents per acre. This sum is provided for under the New Jersey law quoted in the paper.

7. In Panama, the entire sanitation of the Canal Zone costs approximately $2.43 per inhabitant per annum, and the cost per acre is $1.14. (This is about 1% of the cost of the Canal.) The breeding season is twice as long as that in the latitude of New York or Washington, and probably only one-half of the sum expended was for anti-mosquito work. In order to compare the Panama Canal Zone with New Jersey, these costs must be divided by two, to separate the mosquito work from other sanitary work; and by two again, because the breeding season is twice as long in Panama; and again by two, to compensate for the vast difference in rainfall. The result is 31 cents per inhabitant, or 14 cents per acre per annum.

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PAPERS AND DISCUSSIONS

THE SANITATION OF CONSTRUCTION CAMPS.

Discussion.*

By Harold Farnsworth Gray, Jun. Am. Soc. C. E.†

Harold Farnsworth Gray, Jun. Am. Soc. C. E. (by letter).—The writer is pleased with the kindly reception of this paper by members of the Society. It was his purpose to engage the interest of engineers in the importance of this subject, rather than to write a treatise on camp sanitation. For this reason, brevity was considered essential, and the subject matter was confined to a statement of underlying principles, with illustrations of simple methods of applying these principles to practice. The gentlemen who have kindly contributed to the discussion of this paper have described methods and equipment which, if adapted to local conditions, will work as well as, if not better than, those described by the writer, which were given as illustrations simply because they had been tried out under difficult conditions and had proved effective. Mr. Gray.

An oversight which the writer regrets is that due credit was not given for Fig. 2, Plate CXXX, and Plate CXXXI. These photographs were kindly furnished by W. B. Herms, Assistant Professor of Applied Parasitology, University of California, in advance of publication of his recent book, "Malaria—Cause and Control."

Mr. Winsor's discussion brings out several important points. His statement, "under all conditions, incineration [of excreta and urine] is much to be preferred to the latrines suggested by the writer," needs qualification. Restating the fundamental principles of waste disposal, the method is in general satisfactory if (1) during collection the wastes are inaccessible to external agencies (*e. g.*, flies), (2) during collection they do not constitute a nuisance, and (3) they are finally

* Continued from February, 1913, *Proceedings*.

† Author's closure.

Mr. Gray. disposed of in such a manner as neither to create a nuisance nor to permit infection. Given two or more satisfactory methods of disposal, the decision must rest with the most economical method as regards cost of installation and maintenance.

The minimum limitation of air space in sleeping quarters at 400 cu. ft. per occupant has no real practical significance, in the light of recent investigations. The amount of change of atmosphere is a better criterion for the efficiency of ventilation. For example, a space of 100 cu. ft. per occupant completely displaced four times an hour will in general give better results than a space of 400 cu. ft. changed only once. Small ventilator openings near the ceilings of rooms are cheap and effective. In addition, the men should be encouraged to sleep with windows open.

The writer cannot agree with Mr. Winsor that the enforcement of sanitary regulations is a very difficult matter, unless it is difficult to combine tact with firmness in handling men. He has lived in camps for long periods, in close personal contact with laborers of various types, and has reason to believe that they appreciate a square deal, and that a disposition on the part of the contractor to have some regard for their comfort and health will be met more than half way by the majority. Pride in the cleanliness of the camp can be easily obtained. The illustration given by Mr. Payrow's last paragraph is significant.

Mr. Britton has indicated certain legal phases of the subject. In view of some recent decisions, it is now possible for an employee successfully to sue for damages on account of sickness incurred in camp, if negligence on the part of the contractor can be proved. The imperiling of the health of communities adjacent to camps by reason of unsanitary conditions prevailing therein is a matter within the jurisdiction of local and State health officers, and failure to secure proper conditions in this respect is primarily the neglect of duty by public officials, though the contractor can by no means be excused because of a lax enforcement of the law.

In reply to Mr. Austin's query as to screens, it may be stated that bronze and copper screens do not deteriorate as rapidly as iron, especially in moist climates; they are therefore more economical in the long run, and are to be recommended for all except temporary camps. A rusted and broken screen is worse than useless. Mr. Austin's suggestion as to a clause in contracts requiring the contractor to maintain proper sanitation in camps is excellent, and will work to the benefit of contractor and owner alike.

Mr. Payrow's camp layout, though in general excellent, has one defect. It will invariably be better to separate completely the commissary and mess-house from the sleeping quarters by at least 40 ft.; in winter,

a closed passage between the two may be permitted. The use of formaldehyde as described was undoubtedly an extravagance, as it has no effect on flies, either adults or larvæ, as a contact poison, except in high concentrations and at a prohibitive cost. In warm weather, a few dishes of sweetened 3% formaldehyde exposed where the flies tend to congregate will kill large numbers (the flies drink it eagerly). Pyroligneous acid sprayed about places where flies tend to congregate will be found useful as a deterrent; but, if proper attention is paid to the disposal of wastes (excreta, garbage, manure, etc.), flies, as a rule, will be so few that the use of poisons and deterrents is unnecessary. Mr. Gray.

Mr. Payrow mentions the use of disinfectants. The distinction between disinfectants, insecticides, and deodorants should be borne in mind, and an article suited to the intended purpose purchased. In buying disinfectants proper, the manufacturer should be required to give (especially if a proprietary compound) a guaranty as to the potency of the product in terms of the U. S. Hygienic Laboratory phenol coefficient, as determined by an analysis of samples by a reputable laboratory. The disinfectant should be purchased on the basis of unit cost (price per gallon divided by the phenol coefficient), selecting, of course, the cheapest in unit cost thus determined. Such disinfectants should be diluted to be equivalent in strength to about a 3% solution of carbolic acid. If proper cleanliness is observed, however, disinfectants will seldom be necessary.

The writer thanks Mr. Miller for his illustration by a concrete example of the value of proper sanitation in camps. He regrets that more such illustrations were not presented, and considers this an indication that proper camp conditions are the exception.

The spirit of Mr. Hardman's criticism is especially fine. The writer freely admits that he is far more interested, as a citizen, in the altruistic phase of the question; but his endeavor was to make this a paper of engineering interest. He intentionally addressed his argument to the pocketbooks of engineers and contractors. He sincerely hopes, however, that, for the honor of the Profession, the members of this Society will be moved in this matter as much by their obligations as citizens as by any economic benefits to be derived.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS
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IRRIGATION AND RIVER CONTROL IN THE COLORADO RIVER DELTA.

Discussion.*

By W. W. Follett, M. Am. Soc. C. E.

W. W. Follett, M. Am. Soc. C. E. (by letter).—Mr. Cory's paper is valuable and interesting for several reasons. It gives in chronological order the history of irrigation in the Imperial Valley, culminating in the terrible battles with the gigantic and treacherous Colorado; it records the impressions and beliefs, after sober second thought, of one who was there in responsible charge during the months and years of greatest stress, and, above all, it is especially pleasing to the writer because the author takes occasion to show how uncalled for was the hysteria which accused Mr. Rockwood of having made a criminal blunder in opening the cut through which the river made the crevasse.† Mr. Follett.

In March, 1903, the writer made a trip down the Colorado River by rowboat, starting at Yuma and going to a place known locally as "Mike's Landing," which was about 80 or 90 miles by river below Yuma and near the head of tide-water. The return trip was made by wagon, through the delta lands west of the river, along the west bank of Volcano Lake, and to Calexico.

On this trip, which was made on a Yuma gauge of 18.3, or about 1.5 ft. above low water for that season, it was noted that the height of the river banks varied from 5 to 8 ft. above the surface of the water. The maximum flood of 1902 was 24.5 on the Yuma gauge, and, judging from the high-water marks on the trees, filled the river a trifle more than bank-full, whether the banks were 5 or 8 ft. high. Apparently, so much water escaped as soon as the banks were topped that the rise ceased. After passing a point about where the crevasse of 1905 afterward occurred, where the width of the channel was

* Continued from March, 1913, *Proceedings*.

† *Transactions*, Am. Soc. C. E., Vol. LXII, p. 24.

Mr. Follett. 800 ft. or more, the latter narrowed, as water was spilled over the banks, until it was only 410 ft. between the overflow banks at Mike's Landing. These apparently did not overflow deeply—although at that time there had not been for many years such enormous discharges as have come during the last 8 or 9 years.

On leaving the landing, the wagon road led directly away from the river for about 10 miles and then veered to the northward, toward Volcano Lake. The high-water marks showed that there was a rapid fall in the ground as the river was left behind. Of course, these marks were not level, but fell away from the river, as drift showed that the current had been in that direction. The fall mentioned by the author* —from 5 to 8 ft. in the first 3 000 ft.—was found, being nearer the latter figure. After that, the fall was more gradual.

In going north it was noted that each flood-water channel crossed was running on a ridge similar to, but much smaller than, that on which the main river ran.

As a result of that trip, the writer was impressed with the idea that the Colorado was nearing an epochal change in its course across its delta. It is well known that an alluvial stream builds up its banks and beds across its delta until it lies on a ridge; and that finally the time comes when the river breaks away from this ridge, takes a new course to the sea across the lower portion of the delta, and proceeds to build another ridge. When the stream reaches this condition of instability it is impossible, except at great expense, to keep it on the high ground. The writer had no idea, however, that the river would break into the Imperial Valley, but believed that the ridge north of Volcano Lake and the embankment along the canal of the California Development Company, would prevent such a diversion.

Subsequent events have confirmed this idea, and the writer is now of the opinion that any money which may be spent in attempting to close the Abejas Crevasse will be wasted, because, even if it is closed and held, the river will break out again somewhere else.

The author has shown quite clearly† that no serious grade recession has resulted from the Abejas Crevasse. As the river has been running through this opening for 4 years, during which time there have come the two largest floods ever known, it is fair to assume that no further grade recession will occur, but that a new delta is forming, pointing toward Volcano Lake, and that the tendency henceforward will be for the bed of the river to rise. Therefore, no danger to the water supply of the California Development Company nor to Laguna Dam is to be expected. It is best, therefore, to recognize this as an epochal change, allow the river to take its own course to the Gulf, and protect the Imperial Valley by raising and strengthening periodically the levee

* *Proceedings*, Am. Soc. C. E., for November, 1912, p. 13'7.

† *Proceedings*, Am. Soc. C. E., for November, 1912, pp. 1359-1360, and Fig. 3.

Mr. Follett.

which lies north of Volcano Lake and extends back to the river. For the valley this will be a sure protection, which the closing of the Abejas Crevasse would not afford for any length of time.

Attention is called to the narrow gorge between levees, which begins opposite Andrade and extends for 2 miles down stream. The levees average about 1 500 ft. apart, with a width of river channel of 1 000 ft.; that is, the average berm on each side between the river and the levee is only 250 ft., and, in some places, it is probably narrower. This condition is shown on Plate CXVIII.*

If, during some flood, the current of the river should be deflected so as to strike the right bank, and scouring began, it is the writer's belief that the levee and canal bank would be eaten away like sugar, and the river would be diverted into the canal and down the Alamo, again reaching the Salton Sea.

To prevent this diversion, a safety gate can be built about 2 miles below the Hind Dam (see Plate CXVIII*). At this point, the Rockwood Crevasse channel touches the sand hills on the north, and a cross-levee could be built from the gate to the main levee on the south. As there is much fall, a drop would probably be needed. In this might be placed gates, if thought best, which could be used for sluicing sand from the canal above.

The writer was on the ground several times prior to the closure of the crevasse by the Hind Dam, and was strongly impressed with the danger of the river "sideswiping" the canal in this narrow stretch. If it starts to shift, the efforts of man to stop it will seem puny in comparison with the immense destructive force of the current. It is stated that, in the Edinger Dam, piles, 64 ft. long, were driven more than 50 ft. into the bed of the river, and that, when the flood struck them, they scoured out and floated away; they were not broken, but came out bodily. Attention is called to the author's statement that, in 1907 and again in 1909, it was found that for an increase of 10 ft. in the gauge height at Yuma there was a lowering of the bed of about 30 ft.† This was where the river was flowing in a straight and unimpeded channel. It is not possible to say how deep it would scour in a sharp bend. It would be deep enough, however, to undermine and destroy the levee, regardless of any rip-rapping which might be on its face, or any rock which might be hastily dumped into the river. It is the writer's belief that this is a serious menace to the Imperial Valley. The fact that the river channel is now practically straight for about 4 miles is almost proof positive—bearing in mind the idiosyncrasies of an alluvial stream—that it will not remain in that condition very long. Whichever way the bend may start, the current will soon be attacking both sides, as it will rebound from one bank to the other.

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* *Proceedings*, Am. Soc. C. E., for November, 1912.

† *Proceedings*, Am. Soc. C. E., for November, 1912, p. 1359.

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PAPERS AND DISCUSSIONS

A SUGGESTED IMPROVEMENT IN BUILDING WATER-BOUND MACADAM ROADS.

Discussion.*

By J. C. Meem, M. Am. Soc. C. E.

J. C. Meem, M. Am. Soc. C. E. (by letter).—It would appear to the writer that a road built under the author's specifications would undoubtedly have good wearing qualities if it did not become too dry. It is suggested, therefore, that a frequent sprinkling or application of oil would insure its lasting quality in the same ratio that its original value might bear to that of a road built by the alternative method. Mr. Meem.

The presentation of this paper reminds the writer of a piece of macadam road which the author constructed on a private estate in Virginia. About 2 miles of ordinary country road were macadamized, and a highway bridge was built over a tributary of the Shenandoah River. The macadam work consisted of a 12-ft. roadway, which was "metalled" according to the specifications of this so-called improved method, with berms of earth 3 ft. wide on each side, and with under-drains outside of these, as noted later. This road is wearing very well, after 2 years of such traffic as country roads ordinarily get from heavy farm wagons and occasional automobiles; and, although no comparative values are obtainable, its good condition is believed to be due partly to the fact that under-drains, in a trench 12 in. wide and 2 ft. deep, and surrounded with broken stone, were laid to a uniform grade on each side, giving perfect drainage to the highway. The rock used was the native blue limestone, quarried and crushed on the estate.

In connection with this work, the writer designed a reinforced concrete highway bridge, which was built by the author. The question of surfacing the roadway of the bridge was an important one. It was not deemed advisable to carry the macadam over the roadway, as it

* Continued from March, 1913, *Proceedings*.

Mr. Meem. was thought that its tendency to dry out and disintegrate would be much greater than in an ordinary road in contact with earth. The following method was finally adopted: The upper or compression side of the concrete was troweled to a very compact and smooth surface, and, on this, there was placed about 1 in. of lean cement mortar, over which, in turn, from $\frac{1}{2}$ to $\frac{3}{4}$ in. of sand was sprinkled before the mortar had set. The bridge has now been in use about 2 years, and this surface is apparently giving satisfaction. As this method of surfacing was in the nature of an experiment, the writer will be greatly interested in observing from time to time the wearing qualities which it develops.

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PAPERS AND DISCUSSIONS

CONSTRUCTION PROBLEMS, DUMBARTON BRIDGE, CENTRAL CALIFORNIA RAILWAY.

Discussion.*

By Messrs. L. J. Le Conte and R. D. Coombs.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is very much interested in the subject presented in this paper, inasmuch as he was officially connected with the progressive changes made in the proposed plans, and in the final acceptance of the bridge after completion. The local struggles with bad foundations make the construction of the "Dumbarton Cut-off" one of the most instructive pieces of bridgework in the State of California. Mr. Le Conte.

The first trouble encountered was when an attempt was made to put in ordinary trestle approaches from the shore line on each side out to the site of the draw span. These approaches, as well as the draw span, were double-track structures. After a long and expensive struggle, it was found to be impossible to maintain a trestle bridge in the deep and strong tidal currents which prevail at the bridge site. It was finally concluded to put in six 180-ft. through riveted spans, three on each side of the draw span. This caused a great increase in the cost, which was naturally very embarrassing.

The troubles with bad foundations, however, had just begun to develop. For the land approach across the salt marsh from Newark, westward to the bay shore, the plan adopted was to build the embankment by dredging two canals (one on each side), and depositing the material in the embankment lying between. This method led to much trouble in the maintenance of way. The first tracks laid, of course, were only for construction trains, but the embankment settled so irregularly that the track lines could not be kept passable, even for

* This discussion (of the paper by E. J. Schneider, M. Am. Soc. C. E., published in January, 1913, *Proceedings*, and presented at the meeting of March 19th, 1913), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

Mr. Le Conte. them, which means a great deal to one of experience. This trouble continued during the time of the construction of the steel bridge in its entirety. The track lines could not be maintained in serviceable condition, because of the settlement of the embankment and the gradual rising of the beds of the two canals on each side. Finally, the Southern Pacific Company, in desperation, obtained two hydraulic dredges, and put them to work filling up the two canals with good heavy materials, dredged from the bay shore and deposited through two long pipe lines. This expedient had the desired result, and settlement of the embankment practically came to an end. The permanent track lines were then established and well ballasted, and the "Dumbarton Cut-off", after three years' operations, was officially opened to general traffic on September 30th, 1910.

The author's description of the method of putting in the center pier of the draw span is most interesting and instructive. Of course, engineers differ as to the best method of laying concrete under water. The writer is decidedly in favor of the judicious use of the "tremie", as being the most rapid, and therefore the most satisfactory, method. The principle to bear constantly in mind is that, where the face of the freshly laid concrete is being constantly washed by the ponded water, the faster the concrete is laid the less effect the water will have on it. By rushing the rate of laying, the water has not time enough to act on any one face before a new layer is right on top of it, hence, the quicker the laying is done, the better the results.

This principle was first called to the writer's attention in 1872, by the late Gen. B. S. Alexander, Corps of Engineers, U. S. A., who stated that he got it from the late Gen. Joseph Gilbert Totten, Corps of Engineers, U. S. A., Hon. M. Am. Soc. C. E., at Dry Tortugas, off the south end of Florida. The experience, therefore, is of long standing.

The writer wishes to express his profound regards for the late Mr. W. E. Marsh, Assistant Engineer, Southern Pacific Company, who extended many courtesies to him during his official inspections. The Dumbarton Bridge stands to-day as a lasting monument to wise judgment and good construction.

Mr. Coombs. R. D. Coombs, M. Am. Soc. C. E.—The speaker is particularly impressed with the size of the piles used, the lengths being from 60 to 120 ft. and the diameter of the tips either 9 or 10 in. In the East such sizes are not obtainable for ordinary work. The question arises: Was it thought necessary to use large tips in order to increase the bearing power of the piles, a necessity not indicated, at least to the speaker's mind, if the underlying soil was sand and clay?

A certain proportion of the foundation piles extended up into the concrete piers, presumably as an anchorage, and the speaker would in-

quire whether such piles were peeled. In several cases in which the concrete foundations were very shallow, the speaker has extended the piles as far as possible into the concrete, using peeled piles with spikes driven into sides of the embedded portion. The exact anchorage value of this method is not definitely known, but it is undoubtedly considerable. Mr. Coombs.

Referring to the advisability of penetrating so-called crusts, the speaker believes it is usually necessary to do so in pile work, particularly where the "crust" is a vegetable deposit underlaid by sand or clay, or both. In some sections of the New Jersey swamps there is a peat-like deposit about 10 ft. deep, which, if unbroken, might carry a highway load, but is useless for pile foundations. However, the sand and clay below will support heavy loads by skin friction very effectually.

Is it not true that the old formula for the drop-hammer is just as inaccurate as any new formula for the steam-hammer? Is not pile-driving mainly a matter of judgment? In the speaker's opinion, it is.

As an illustration, two more or less typical experiences may be cited: In one, the foreman informed the speaker that he had just driven a spliced pile 190 ft. long. Naturally, he was at once assured that he had not done anything of the kind. The fact was that, through a conscientious attempt to obtain the usual penetration per blow, the piles were being broken at the splices and driven horizontally. Hard bottom was not more than 40 ft. below the surface.

In the second case, piles from 65 to 75 ft. long, driven in blue clay, for a wharf, were stopped under easy driving to prevent them from going below cut-off. A few months later, stevedores unloaded 80-lb. rails on the wharf, piling them 6 ft. high, and there was no settlement.

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PAPERS AND DISCUSSIONS

SHEARING STRENGTH OF CONSTRUCTION JOINTS IN STEMS OF REINFORCED CONCRETE T-BEAMS, AS SHOWN BY TESTS.

Discussion.*

By L. J. Mensch, M. Am. Soc. C. E.

L. J. Mensch, M. Am. Soc. C. E. (by letter).—These tests present scientific proof of facts which are well known to the experienced worker in reinforced concrete, but are often questioned by engineers and architects. They prove clearly that joints in reinforced concrete construction can be made as strong as a monolithic piece of work, provided they are amply reinforced. It is to be hoped that they will be studied by engineers and architects, and that a study of this kind will prevent such an experience as happened to the writer some years ago in Salt Lake City: He was constructing some girders of 52-ft. span, and the architect compelled the men to finish the girders and slabs in one operation, making them work 20 hours at a stretch and thereby disorganizing the whole gang for a week. Mr. Mensch.

These tests also demonstrate clearly the great influence of stirrups on the strength of reinforced concrete T-beams.

Beams of Type *C*, having the same longitudinal reinforcement as those of Type *A*, but not as many stirrups, show, in the average, 17% less strength than the latter. In the calculation of these stirrups, it will be of interest to apply the rules recommended by the Joint Committee. The area of both legs of a $\frac{5}{16}$-in. round stirrup is 0.153 sq. in., and that of a $\frac{3}{16}$-in. round stirrup is 0.0553 sq. in. The safe shear in these beams will be assumed to be one-eighth of the total load given in Table 2, and amount to 6 350 lb. for beams of Type *A* and 5 380 lb. for beams of Type *C*. The Joint Committee recommends

*This discussion (of the paper by Lewis J. Johnson, M. Am. Soc. C. E., and John R. Nichols, Jun. Am. Soc. C. E., published in February, 1913, *Proceedings*, and presented at the meeting of April 2d, 1913), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

Mr. Mensch. the use of only two-thirds of these values in the formula; they are, therefore, 4 230 lb. in Type A and 3 580 lb. in Type B; assuming $j\,d = 11$ in., then the spacing of the stirrups for Type A equals $\dfrac{16\,000 \times 0.153 \times 11}{4\,230 \text{ lb.}} = 7$ in., and for Type B $\dfrac{16\,000 \times 0.0553 \times 11}{3\,580 \text{ lb.}} = 3$ in., which is not a very good agreement.

From other tests, the writer has reason to believe that a beam with the same longitudinal reinforcement, but without stirrups, would have failed at a load of about 38 000 lb., or at one corresponding to a safe shear of 4 750 lb.

Assuming that the excess of shear is taken up by the stirrups, then the stress in beams of Type A equals $\dfrac{(6\,350 - 4\,750) \times 5.5}{0.153 \times 11} = 5\,230$ lb. per sq. in., and in beams of Type B, $\dfrac{(5\,380 - 4\,750) \times 5.5}{0.0553 \times 11} = 5\,700$ lb., which is a better agreement, and shows that the stirrups in Type B are more highly stressed than those in Type A, although these stresses are probably fictitious.

According to the authors, the reinforcing bars were probably cold twisted. In the writer's opinion, however, they certainly were cold twisted, and are equivalent to high-carbon steel bars.

According to the Chicago building ordinance, a stress of 18 000 lb. is allowed for this grade of steel. It is very important to note that Type A beams failed at a stress of 58 750 lb., showing a factor of safety of only 3.25, and Type C beams at 51 650 lb., showing a factor of safety of only 2.87, in bending. On the other hand, tests made by Professor Schüle, of Zürich, Switzerland, and by others, prove that concrete beams with low end shear, and reinforced with steel of this grade, have failed at a stress in the steel of about 72 000 lb. per sq. in., and some tests with a higher grade of steel, show a stress as high as 140 000 lb. per sq. in. This poor showing of the longitudinal reinforcement, in the case of beams with high end shear, is confirmed by a great number of practically identical tests of monolithic beams made by Professor Bach, of Stuttgart, in 1911. In order to gain a high resistance to shear, therefore, we have to sacrifice a high resistance to bending.

It is also very important to note that the weight of the stirrups in beams of Type A, with a stem width of only 4 in. and a total height of only 15 in., is 2.2 lb. per lin. ft. of beam, or 33% of the weight of the longitudinal reinforcement. If tests had been made on beams with a stem width of 16 in. (such beams often occurring in practice), the weight of the stirrups would have amounted to 9 lb. per lin. ft.

Stirrups cost very much more per pound in place than longitudinal

reinforcement, very often from 50 to 100% more, and it is clear that it is cheaper to increase the width of the stem than to provide such an excess of stirrups. Mr. Mensch.

The writer cannot agree with the authors' conclusion, that an allowable stress of 125 lb. in shear is low enough. On the contrary, he considers it a limit which should only be adopted very rarely, and then only by experts, as it leads to uneconomical and dangerous designs. This high resistance of end shear could not have been obtained if the anchorages of both straight and bent bars had not been unusually well designed, which is rarely done in practice. Ordinary hooks, 3 or 4 in. long, would never have given these high results.

It is also evident that the substitution of plain, round, high-carbon bars for the twisted bars, would have somewhat delayed the splitting of the beams at the ends.

The tests of beams of Type *B* are certainly surprising. It must be kept in mind, however, that the stress of the longitudinal reinforcement is very low, that the weight of the stirrups is 66% of that of the longitudinal reinforcement, and that their cost is certainly greater than that of a corresponding increase of width of the web. Professor Bach also made a great number of tests of beams similar to Type *B*, but obtained shearing stresses of only 550 lb. per sq. in.

In the tests made by Professor Bach, and more so in the tests described in this paper, especially for beams of Type *B*, the span is unusually small. It is well known that the results for resistance to shearing and bond of the concrete to the steel rods are more favorable in beams of small span, and the high shear of 880 lb. could never have been obtained in a test beam 20 ft. long.

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PAPERS AND DISCUSSIONS

FREMANTLE GRAVING DOCK: STEEL DAM CONSTRUCTION FOR NORTH WALL. Discussion.*

By A. R. Archer, Assoc. M. Am. Soc. C. E.

A. R. Archer, Assoc. M. Am. Soc. C. E. (by letter).—The work described by Mr. Ramsbotham is noteworthy by reason of the unusual conditions surrounding the driving of the steel sheet-piling, the varied character of the ground through which it was driven, and the extreme penetration obtained on the shore side, that is, at the back of the embayment for the pumping station. Mr. Archer.

In designing the bracing the author evidently desired to use a uniform size of timber throughout, with as little framing as possible, and, had it been necessary to brace the dam for the full depth of the diagram shown in Fig. 3, the spacing of the wales near the bottom would have been too close, as far as the strength of the steel skin was concerned. Within the limits to which excavation actually progressed, the spacing approximated very closely to that which the manufacturers of the piling recommended, but at greater depths this, necessarily, depends more on the character of the ground than on any theoretical considerations. The details of the bracing are in accordance with modern American practice, but the hardness of the jarrah wood used evidently permitted the omission of short foot-blocks at each end of the shores, in the lower tiers of bracing, such as are commonly used in the United States to reduce the span of the wales and prevent the end of the shore cutting into the face of the waling, something which the writer has observed several times in relatively deep coffer-dams in which these foot-blocks were omitted. The bracing as

* This discussion (of the paper by Joshua Fielden Ramsbotham, Assoc. M. Am. Soc. C. E., published in February, 1913, *Proceedings*, and presented at the meeting of April 2d, 1913), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

Mr. Archer. a whole was designed for water pressure, but, from the conditions shown in the diagrams, that portion of the coffer-dam forming the shore side of the embayment for the pumping station was subjected to earth pressure, and it would be interesting to learn whether there was any indication of this exceeding the calculated water pressure. That something of the sort was anticipated is shown by the use of the concrete anchors with wire cable ties shown on Figs. 2 and 4.

The author has made several suggestions to the manufacturers of the steel sheet-piling, and in justice to the latter it should be stated that the pamphlet* published by them in 1912, covers most of the points to which attention has been drawn. A copy of this pamphlet is in the Library of the Society, and therefore available for reference by members.

In view of the large area to be unwatered—10 453 sq. ft.—and the severe leakage from the bottom, one might assume that leakage through the joints of the steel sheet-piling was not a serious factor; some information on this point would be of value. It is also to be assumed that the rapid corrosion of the steel by the discharge of sulphureted hydrogen would rapidly check any leaks which might exist.

The author should have the thanks of the entire Profession for his very careful description of grouting the bottom, as this is a subject on which but scant information can be gleaned from published engineering works.

* "Steel Sheet Piling: Tables and Data on the Properties and Uses of Sections."

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PAPERS AND DISCUSSIONS

CHARACTERISTICS OF CUP AND SCREW CURRENT METERS PERFORMANCE OF THESE METERS IN TAIL-RACES AND LARGE MOUNTAIN STREAMS STATISTICAL SYNTHESIS OF DISCHARGE CURVES.

Discussion.*

By Messrs. C. W. Staniford and B. F. Groat.†

Mr. Staniford.

C. W. Staniford, M. Am. Soc. C. E.—The speaker would like to ask Mr. Groat or Mr. Hoyt whether any of the current meters mentioned—the Haskell, the Price, the Ellis, or perhaps the admirable German instruments—after being rated in fresh water (still water, of course), have been used in salt water; and what percentage of accuracy can be obtained with such meters in salt water?

If such instruments have been rectified against corrosion, and against short-circuiting, can they be relied on when used in salt water?

Mr. Groat.

B. F. Groat, Assoc. M. Am. Soc. C. E. (by letter).—If one will examine the papers and discussions concerning current meters published since the original study was made by Frederic P. Stearns, Past-President, Am. Soc. C. E., in 1879,‡ it will be found that there is much uncertainty among engineers concerning the reliability of these instruments, with which, up to the present time, there does not seem to have been any generally satisfactory and generally adopted practice. These facts are in sharp contrast to those connected with the use of other engineering instruments, such as the transit and level. No method can be deemed generally satisfactory until it meets the general approval of

* Continued from February, 1913, *Proceedings*.

† Author's closure.

‡ *Transactions*, Am. Soc. C. E., Vol. XII, 1883, p. 301.

Mr. Groat. practitioners. So long as there is lack of agreement between various classes of current-meter observers, not only as to the general method of procedure, but also as to the reliability of various types of instruments, there can be no such thing as a satisfactory current-meter practice.

When it becomes generally realized that current meters obey their still-water ratings only when the conditions of flow are ideal, and that cup meters are affected in one sense and screw meters in the contrary sense, it will be apparent that there is a real reason for the diversity of opinion concerning the reliability of current-meter work.

The writer, therefore, was inspired by the hope that this paper might reopen the discussion and possibly put in motion an effort to improve the current meter.

Appended to the paper there is a brief description of a new system of taking discharge observations and a new method of constructing the discharge curve. This system and method have been applied most successfully in a number of cases, and are worthy of the attention of those contemplating a series of gaugings at one or more stations.

The method of synthesizing a discharge curve is not an invention of the writer which he is trying to introduce, but is a well-established method used by physicists and practicing engineers in determining various kinds of relations between observed magnitudes. It consists simply in separating a given investigation into its elementary parts and drawing a curve for each, the final relations being derived from the elementary curves or relations. The fact that the sum of the velocities at a given set of meter points at a gauging station forms such an elementary relation, when plotted as a function of the gauge height, leads to the term "statistical."

The main advantage of the statistical method is that the entire series of gaugings, extending over a long series of years, may be connected into a consistent whole. The peculiar characteristics of a station may be studied from the elementary curves. If any change or break occurs in one or more of the elementary curves, a corresponding change or break has taken place in one or more of the physical characteristics or hidden conditions of the station. This is especially true of the sum of the velocities at a given set of meter points, because the sum forms a fixed relation between velocity and stage. The sum is chosen because, in taking a number of velocities, the accidental variations and errors in the velocities eliminate themselves. That is why, even at turbulent stations, a smooth curve can be drawn through the plotted values of the sum, with errors well within 1 or 2%, as extremes. It is not likely that changes in the hidden conditions at a station would be detected by any other method. They would be set down simply as unaccountable variations.

Another very important advantage of the statistical method is that it affords at once the most independent and most easily secured check on the velocity readings. Indeed, the sum of the velocities must plot on a definite curve, and this enables one to check the meter work at any newly observed stage with that at any other stage within 10 min. after all the velocities for the former have been recorded. Mr. Groat.

The writer is glad to know that his Conclusion 1, concerning cup meters, is generally recognized. Under certain nearly ideal conditions, such meters will give good results, but, the moment the conditions are changed in the slightest degree, the still-water rating becomes more or less inapplicable, with a resulting constant or systematic error.

Nevertheless, it would be perfectly safe to use a cup meter in turbulent water if, where used, it were rated with a Pitot tube. The fact that the meter runs with fidelity under such circumstances is another of the conclusions drawn in the paper, and one should not lose sight of this good quality. It may also be remarked that this is a fact which does not seem to have been generally recognized. Cup meters will undoubtedly have their uses, but improvements in the means for obtaining their ratings—not still-water ratings, but actual ratings at the time and place of use—are imperative. It will do to talk about refined ratings when probable errors are reduced to values of less than from 3 to 5 per cent.

After reading the discussions one can scarcely shake off the idea that the niceties demanded for the conditions under which cup meters must be rated are altogether out of proportion to the degree of accuracy attainable with these meters in stream gauging. If a cup meter cannot be rated accurately from a boat when operated carefully, owing to changes in rating due to the slight irregularities of motion, what must be the effect on such a meter when subjected to the far greater irregularity of the motion of the water at a fairly good gauging station, to say nothing of the errors to be expected at a really turbulent station.

The writer believes that Mr. Miller made an important discovery in current-meter engineering when he noticed that cups over-register considerably and screws under-register slightly in agitated water. In due time this fact will impress itself on engineers generally, and lead to a meter which obeys its still-water rating more closely than any hitherto made.

The Haskell meter is so nearly correct under a wide range of conditions that it would seem to require only slight modification, as suggested in the paper, to render it almost independent of conditions. It would be a delightful task, for some one who has the time and means, to develop such a meter. Meantime, it cannot be too strongly urged on hydraulic engineers that a screw current meter is the

Mr. Groat. most reliable kind for measuring the velocities of flowing water when using the still-water rating.

It may be well to define the terms "cup" and "screw," as the writer does not wish to apply the general propositions stated to every kind of instrument which may chance to be called a cup meter or a screw meter: A cup meter is one in which the rotating element operates on the principle of the Robinson cup anemometer, though the shape of the cups need not be limited to that of the hemisphere. A screw is a rotating element the surface of which would be generated by a straight line perpendicular to, rotating uniformly about, and moving uniformly along, the axis of rotation.

One would also hesitate to apply the propositions to meters having large or badly designed frames which would be likely to interfere with the action of the water on the runner. Although the frame of the cup meter is at the rear of the runner, it has decided and different effects on the rating of the instrument when turned somewhat to the right and then an equal angle to the left of the line of motion.

The writer is more than pleased to have Mr. Price's open-minded discussion. The cup meter is very ingenious, and the means which Mr. Price has adopted to protect the bearings and secure uniformity of friction have probably been the cause of the continuance of this meter in the Government service.

Mr. Price may be assured that the meter was suspended in precisely the same manner during the turbine tests as in the ratings. This applies also to the stream tests. No weights were used in any case, and the meter wheel was about 7 ft. in advance of the stem of the rating boat. The reaction of the boat on the screw must have been slight indeed, as the ratings from a car at the Rensselaer Polytechnic Institute do not differ materially from those by the boat.

Fig. 1 shows the rating boat, and below it the rod used to support the meter. The hooks on this rod may be adjusted so that any desired series of depths may be secured. From the staging on which the observer stood a $\frac{5}{8}$-in. rod extended across the tail-race at a fixed elevation. The meter was attached to one end of a $\frac{3}{4}$-in. rod, about 2 ft. long, the other end of which was screwed into the supporting rod. There were seven hooks on the supporting rod during the turbine tests, any one of which could be hooked over the $\frac{5}{8}$-in. rod on the staging, thus immersing the meter to the corresponding depth. Six rope clips were attached to the $\frac{5}{8}$-in. rod at the verticals in which the meters were to be operated. The rope clips served as stops at which to hold the meter rod, when taking observations in the verticals.

Fig. 2 is a rough sketch of the Pitot tube and supporting rod. The Pitot tube (to the right) is a copy of "Tube *N*," as described in Mr. White's monograph, referred to in the paper. The supporting

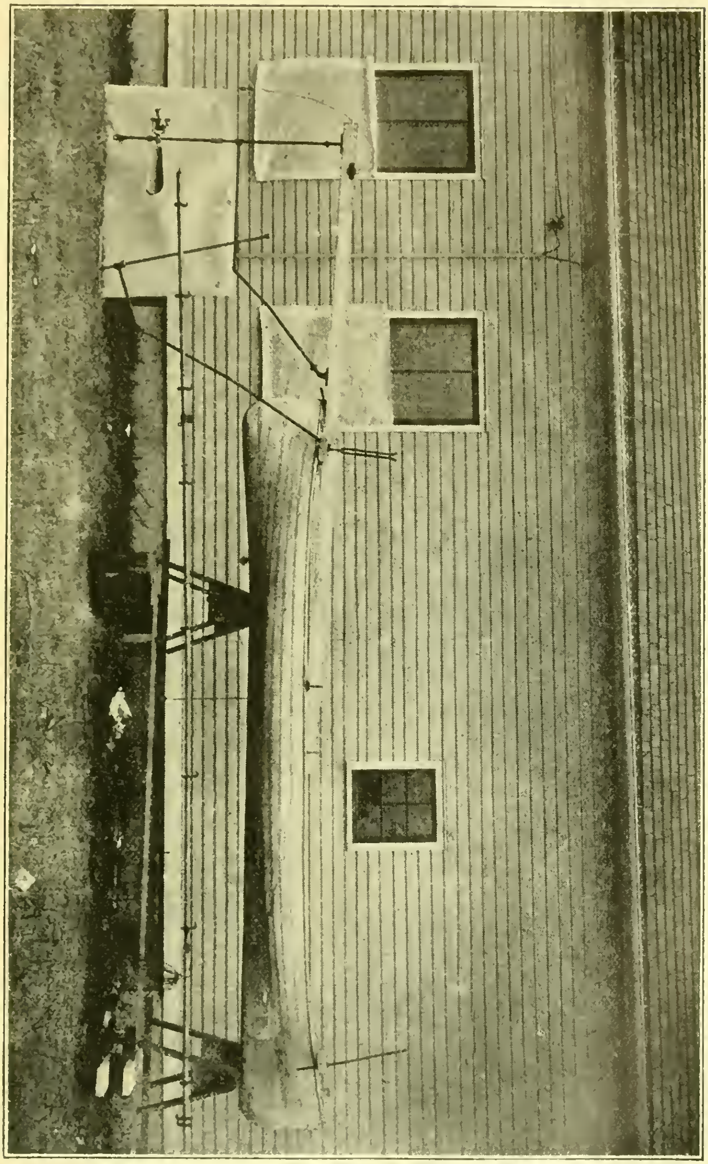

FIG. 1.—RATING BOAT, WITH ROD USED TO SUPPORT CURRENT METER.

rod (to the left) was designed to be operated from the staging in the tail-race, in the same manner as the supporting rod for the meters. On Fig. 2 is shown one of the hooks to engage the ⅝-in. rod on the staging, and also the eyes on both the Pitot tube and meter supporting rods to which a guy wire was attached near their lower ends, thus holding the rods and preventing bending. Mr. Groat.

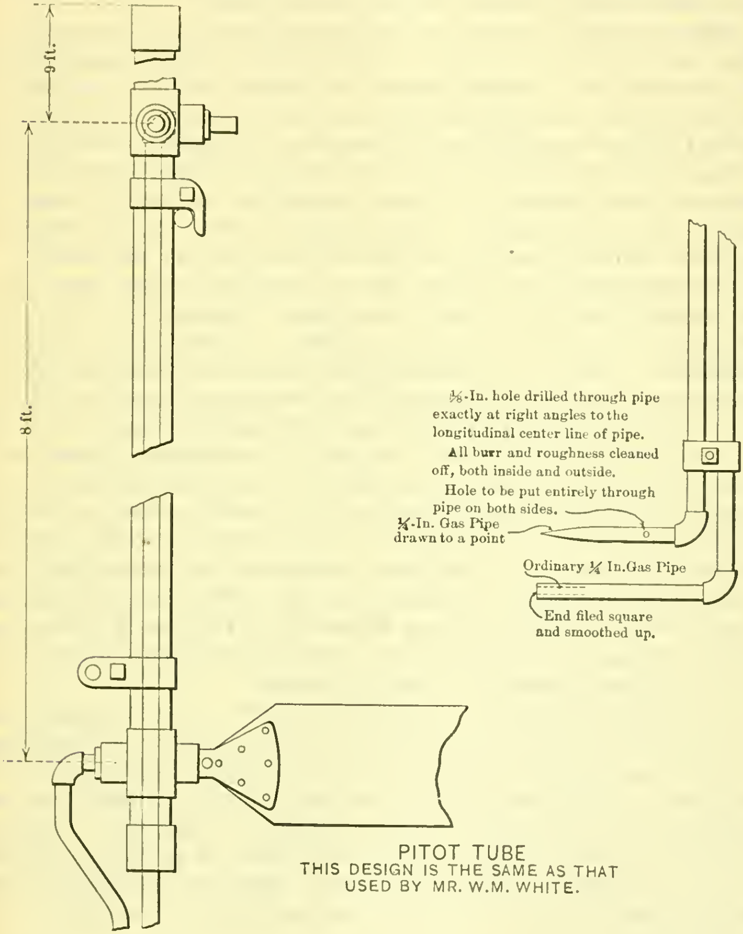

Fig. 2.

Mr. Price states that the meter of the propeller type does not free itself from such débris as dead grass and leaves. This is too broad a statement, for there is at least one meter of that type which does free itself, and that is the Haskell. This meter has been used by the writer

Mr. Groat. many times in streams carrying much dead grass, leaves, twigs, sand, and mud. On no occasion did it catch the slightest particle or give any evidence of being affected thereby.

This is due to the effective shape of the runner, which Dean Haskell has undoubtedly designed expressly for the purpose. The up-stream end of the wheel is brought down to a moderately pointed or conical nose which sheds all débris carried by the water.

The question of friction in a current meter is very interesting. Throwing out the case of low velocities, the necessity seems to be, not so much for a small amount of friction as for constancy of amount. If the resisting torque is large but constant, a rating should be fixed just as exactly as in the case where the friction is small and constant. Notice, too, that there is a resisting torque caused by the hydraulic skin friction on a screw, which can be made as large as desired by lengthening the vanes along the axis. In this respect, the screw is just as much a differential meter as the cup meter.

The retarding skin friction has a much longer lever arm than the bearing friction, and has a much larger resisting torque, so that the bearing friction of a screw meter offers a resistance of secondary importance. Further, the resisting skin friction is a fixed function of the velocity, varying approximately as the square thereof. Hence it is clear that the principal portion of the resisting torque is fixed for each velocity and cannot be changed. It follows that, if the meter is kept in only fair condition, the rating is not affected seriously by moderate changes in bearing friction. The truth of this was tested in the experiments at Massena by taking part of a rating while the bearing was dirty and stiff, then completing the rating after the meter had been cleaned and oiled. Let the reader select from ratings *E* and *F* of the Haskell meter, Plate CXXXIV,* the points taken before cleaning and oiling the meter. The Haskell meter is not sensitive to moderate changes of bearing friction.

Mr. Price thinks the ratings (Plate CXXXIV) of the Haskell meter are inaccurate. If any one will take the pains to examine the percentage errors of the various ratings and the wide range of conditions under which these ratings were made, he will feel compelled to admit that the performance of this meter was very consistent throughout. Ratings of this kind are of more practical value than those obtained under restricted conditions, because the former afford an indication of the degree of accuracy to be expected of the meter when in actual service, while the latter do not.

No doubt the rating line could be adjusted to the observations somewhat better. This rating line was located from a few of the earlier ratings, the later ones being plotted on the diagram subse-

* *Proceedings*, Am. Soc. C. E., for December, 1912.

quently. The change, if made, would be trifling, excepting for low velocities, which were not used. Mr. Groat.

The idea must not be entertained that the conditions in the Massena tail-races were bad. On the contrary, they were much better than those usually encountered in stream gauging. The flow of the water was parallel to the side-walls, and the meters might as well have been fixed rigidly to the rods, as far as concerned any tendency to swing. The errors of the meters were caused by small eddies and boils such as would certainly have to be dealt with at most gauging stations. The conditions of flow were carefully studied at each of the 84 meter points by using an inverted weather vane. An instrument was invented and used to ascertain the quantity of air in the water at each meter point. This quantity was found to be negligible.

Mr. Price states that a gauging station which has an up-stream current should never be used, and one which has divergent currents cannot give accurate results with any method of measurement. It may be that gauging stations with negative velocities should not be used, but do our current meters give any indication which would lead one to discard such a station when encountered? If the meter is 4 or 5 ft. under water, it cannot be seen; but it goes on recording the velocity just the same, even if the current is directed up stream, and the unsuspecting observer records a considerable positive velocity at the point, whereas it should have been recorded as negative.

As to the matter of divergent currents, the writer believes most fully that the problem may now be solved by using a screw meter held rigidly at right angles to the direction of flow. When the screw is modified, as suggested in the paper, it will increase still further the reliability of the results, even at the best of gauging sections.

Dean Haskell says: "Although a great deal of good work has been done with meters of this type [meaning cup meters], they have not shown themselves to be anywhere near as reliable as those of the screw wheel type." This seems to describe the case exactly. Where conditions have been favorable, the errors of the cup meter have not been large; but the errors are there, and they are of one sign.

Though the writer has been compelled to adopt the method suggested of observing the direction as well as the velocity, when gauging large rivers, he has found it fraught with difficulties, the principal one being that he has never been able to find a cross-section on a large river where the direction at each meter point was fixed. The meter will swing, and this completely masks the average true direction. If the meter was held rigidly, and was designed to give the resolved component, the difficulty would be obviated instantly, and the observations and computations would be reduced in number.

As the Haskell meter as now made gives only a trifle less than the resolved component, the writer takes the liberty to suggest that

Mr. Groat. a slight modification, as described in the paper, might render it absolutely exact in perturbed water. A mechanical advantage to be gained by "cupping" the wheel vanes would be that of additional strength for a given weight.

The Haskell meter has one other advantage which has not yet been mentioned: The screw, or propeller, may be made exceedingly light, so that the effects of inertia are reduced to a minimum. It is quite possible that the inertia of a heavy revolving part, in conjunction with the reaction of the water, may play an important rôle in changing the rating when the velocities become variable.

The still-water rating is not satisfactory at present because meters must be used in water which is more or less perturbed. Otherwise, the current meter becomes an instrument of limited use, and demands attention to a complicated system of details. As soon as a meter is produced which will resolve the momentary velocities at a meter point, the still-water rating will apply, even in perturbed water. Dean Haskell is to be congratulated on the fact that his meter almost does this very thing, so that probably only slight modification will be necessary to secure the desired result.

The float and meter comparisons are interesting. Applying the statistical method of summing the velocities in each case, the resulting comparison follows:

Sum of velocities by float	19.67
Sum of velocities by meter	19.74
Difference	0.07

Mr. Miller's discussion is very interesting, as it recounts work of unusual magnitude on the Mississippi. The writer has lived at various points on that river all his life, and remembers much of the Government work of various kinds there.

It is to be regretted that such a keen observer as Mr. Miller should have left the field of current-meter engineering. However, a useful fact once established is bound to live, and thus Mr. Miller is still potentially active in this branch of science.

Mr. Hoyt seems to fear that an impression may be drawn from the paper that the Geological Survey records, based on measurements with the small Price meter, generally involve large errors. This fear is hardly justifiable, as the writer states very definitely, "it seems certain that the correction for a cup meter when run at a good meter station on an open river is not large." On the other hand, it would certainly be on the side of safety to determine the degree of error in every important series of gaugings. One of the best ways to uncover an error of this kind is to run a cup meter and a screw meter in the section simultaneously. If the meters agree, there is no error. If

they disagree materially, it will be found that the velocities by the cup meter are excessive, and those by the screw are deficient, the numerical value of such errors by the cup meter being relatively larger than those corresponding by the screw meter. Mr. Groat.

If any one wishes adminicular evidence as to the truth of this statement, beyond that afforded by the paper, he may refer to the paper, "Current Meter and Weir Discharge Measurements,"* by E. C. Murphy, M. Am. Soc. C. E. In this paper Mr. Murphy has, in the clearest manner, plotted the results of a number of comparisons of discharges by a weir with simultaneous discharges by cup and screw current meters.

In the case of the cup meters, practically all the discharges exceed the corresponding discharges by weir. This is indicated by the fact that practically all the points for the cup meters plot below the axis of abscissas in Fig. 2 of that paper.

In the case of the screw meter, the discharges are generally deficient, as the points generally plot above the axis of abscissas. Moreover, this is the more accurate meter, because the points average much closer to the axis, being in fact within 0.07% of it.

Mr. Hoyt concludes his discussion with several numbered paragraphs. They may be taken up in the same order:

1.—The paper does much more than call attention to the recognized fact that the cup meter is affected by cross-currents. It calls attention to the fact that the effect is productive of errors all on the side of excessive velocity, and that the screw meter is affected in the contrary sense, but to a relatively smaller extent. Though this was discovered by Mr. Miller years ago, it does not seem to have been recognized by anybody else since that time; but it is undoubtedly of the greatest importance to those who wish to improve the present methods of stream gauging.

Before carrying out any experiments at Massena, the writer was of the opinion that screw meters, like cup meters, would always show excessive discharges. This, he thought, would be the case because of the swinging of the meter and the consequent measurement of the thread of a sinuous, rather than of a straight, current. This, however, is incorrect. To prove it, attach a screw meter to the rating boat or rating car. Draw the meter over its course in the usual manner and count the revolutions. Then repeat the process, having all conditions the same as before, except that the meter is oscillated transversely to the direction of motion during its transit of the course. If the rating boat is used, the oscillation may be effected by rocking the boat. In the case of the car, some mechanical device may be used.

Theoretically, the number of revolutions in the latter case should exceed those in the former case. Practically, the reverse is the case.

* *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 376.

Mr. Groat. The side-wash of the current has somewhat more than annulled the effect of increasing the length of the course.

2 and 3.—It is true that the methods used by the writer in making his discharge measurements were described only in a general way. Otherwise, the paper would have become so voluminous that the main facts would have been obscured. It must be remembered that the conclusions of the paper are not derived from a study of current meters *per se,* but are inductions from observations on the behavior of meters while testing turbines.

4.—The large Price meter was adopted as one type of meter to be used in the Massena tests, because the same type had been used by other engineers in the same tail-races several years before, and a comparison was desired. The results of the turbine tests would have been the same had the small Price, or any other cup meter been used instead of the large Price meter, because means were taken to correct the errors due to deviations from the still-water ratings.

5.—The standard adopted by the writer when comparing the cup and screw meters was the true velocity of the water. This appears when discussing Table 5, where it is shown that the cups over-register, on the average, by 5.6%, and the screw under-registers, on the average, by 0.93 per cent.

When the Pitot tube and Haskell meter were compared, the Pitot tube was supposed to indicate, on the average, the true velocity. The result of this comparison was that the Haskell meter under-registered by about 0.6 per cent.

Thus there were two different methods of determining the discharge: (*a*) by cup and screw meters corrected through the medium of the boat-rocking tests, and (*b*) by Pitot tube and screw meter comparisons by which corrections were determined to be applied to the metered discharges.

It is worthy of note that these two methods of correcting the discharge, when applied to any given turbine test, give rise to a discrepancy of scarcely 0.3 of 1 per cent.

It is not claimed that rocking the boat is the best means for ascertaining the relative errors of cup and screw meters. It is claimed that the nature of the case is such that extreme accuracy in this respect is not necessary. This is because the errors to be determined are not relatively large.

If it is feared that this paper is too daring in its conclusions to be credible, it may be well to search for independent verification. Such material may be found in Mr. Murphy's extremely valuable paper, "Current Meter and Weir Discharge Measurements," previously quoted.

Though Mr. Murphy gives an interpretation of the results of his experiments, yet he expressly states that he has "avoided in this paper

a detailed comparison of results by different meters." Hence it will be permissible to analyze the results for the purpose of such a comparison. Mr. Groat.

The best method of eliminating accidental errors in comparing such a series of observations is the "statistical" method. In the case at hand, this method is particularly facile because comparable results may be obtained by the simple operation of summation.

In order the better to understand the conditions under which the tests by Mr. Murphy were conducted, it may be stated that they were made in the canal of the Hydraulic Laboratory at Cornell University. The discharge in each test was measured by weir and by two current meters. The two current meters were observed simultaneously in the same discharge section, one starting on the south side and progressing north while the other started on the north side progressing south.

There were twenty-five experiments, each made in the manner described. Ten of these were comparisons between the weir, Haskell meter No. 3, and small Price meter No. 363. Nine were comparisons between the weir, small Price meter No. 351, and small Price meter No. 363. Six were comparisons between the weir, Haskell meter No. 3, and small Price meter No. 351. Moreover, from these experiments may be selected sixteen in which the Haskell meter is compared with the weir, fifteen for Meter No. 351, and nineteen for Meter No. 363.

There are, therefore, six sets of experiments which may be compared statistically. In each case the aggregate discharge by the weir may be compared with the corresponding aggregate, or aggregates, by the meter, or meters, as the case may be. Table 9 gives the comparisons in tabular form.

The function of such a summary of aggregates is to eliminate in large measure the unavoidable accidental errors arising during the observations. Thus the remaining errors are each composed of two parts, namely, a large part, which is the net average constant error of the observations, and a small part, which is the net residual of the unequated accidental errors. From the approximate uniformity of the errors of the table, it may be concluded that accidental errors have been practically eliminated, and the computed errors for the sets of observations of greatest weight (the first three sets in the table) are the constant errors due to the meters, or due to deviations from their still-water ratings. Thus the constant error of the Haskell meter is — 0.02%, that of the small Price No. 351 is + 1.94%, and that of the small Price No. 363 is + 1.24 per cent. It may be inferred from the plottings by Mr. Murphy, on page 376 of his paper, that the average constant error of No. 351 should exceed that of No. 363. The reason for this is that the discharge by either of the meters shows an increasing excess over the discharge by the weir with

Mr. Groat.

decreasing velocities, and there are relatively more observations of low velocity by Meter No. 351 than by No. 363. Had the reverse been the case, it might have been expected that the constant error of No. 363 would exceed that of No. 351.

TABLE 9.—STATISTICAL SUMMARY OF TWENTY-FIVE EXPERIMENTS BY E. C. MURPHY, M. AM. SOC. C. E., IN WHICH THE DISCHARGE OF A CANAL IS MEASURED SIMULTANEOUSLY BY A WEIR AND TWO CURRENT METERS, THE LATTER OPERATED IN THE SAME DISCHARGE SECTION.

Compiled from Table 1, *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 373.

No. of experiments.	Sum of all the discharges in all experiments, as shown by weir, in cubic feet per second.	SUMS OF THE DISCHARGES IN ALL THE EXPERIMENTS, AS SHOWN BY THE VARIOUS METERS.								
		Screw Type.			Cup Type.					
		Haskell Meter.			Price Meter No. 351.			Price Meter No. 363.		
		Sum, in cubic feet per second.	Error, in cubic feet per second.	Error. Percentage.	Sum, in cubic feet per second.	Error, in cubic feet per second.	Error. Percentage.	Sum, in cubic feet per second.	Error, in cubic feet per second.	Error. Percentage.
16	3 474.78	3 473.99	— 0.79	— 0.02	....	.....	....	....	.....	....
15	3 373.67	....	....	....	3 439.03	+ 65.36	+ 1.94	....	.....	....
19	4 186.17	....	....	....	....	.....	....	4 238.23	+ 52.06	+ 1.24
10	2 143.64	2 147.39	+ 3.75	+ 0.18	....	.....	....	2 164.22	+ 20.58	+ 0.96
6	1 331.14	1 326.60	— 4.54	— 0.34	1 368.26	+ 37.12	+ 2.79	....	.....	....
9	2 042.53	....	....	....	2 070.77	+ 28.24	+ 1.39	2 074.01	+ 31.48	+ 1.54

It will be seen that the error of the Haskell meter for the ten comparisons with weir and small Price meter No. 363, is + 0.18 per cent. This error, though insignificant, would have been negative had it not been for three experiments, Nos. 1-2, 3-4, and 5-6, in which this meter, for some unaccountable reason, gave discharges 4 or 5% greater than the weir. These three observations are plainly discordant, as may be seen by referring to the plottings for the Haskell meter in Fig. 2 of Mr. Murphy's paper. Had the meter run in these three experiments according to the same law as in all the other experiments, the errors for these three experiments would have been about nil. Thus the excess in discharge by the Haskell meter in the three experiments in question is about 25 cu. ft. per sec., as may be seen by consulting Table No. 1 of Mr. Murphy's paper. The aggregate for the weir from the writer's comparison, Table 9, is 3 474.78, and that for the meter would become 3 473.99 — 25.0 = 3 448.99. Thus the deficiency in the aggregate for the Haskell meter would be 3 474.78 — 3 448.99 = 25.79. In other words, it is more than probable that the discharges by the

Haskell meter are deficient by the slight percentage of 25.8 ÷ 3 475 = 0.75. However, it will not do to correct or reject observations simply because they are discordant. Mr. Groat.

It follows from the results shown in Table 9 that the cup meters in all cases gave excessive discharges and the Haskell meter gave relatively much smaller errors, the discharges being slightly deficient on the average. Therefore, the following general conclusions may be drawn from the results contained in Mr. Murphy's valuable paper:

(*a*) When a cup meter is run in moderately perturbed* water, it will register a larger number of revolutions per second than a perfect still-water rating would indicate.

(*b*) When a screw meter is run in moderately perturbed water, it will register a smaller number of revolutions per second than a perfect still-water rating would indicate.

(*c*) In the foregoing sense, a cup meter is affected relatively to a much greater extent than a screw meter.

(*d*) Either type of meter, when run in moderately perturbed water, will give uniform records in equal times, provided these times are sufficiently long, the flow of the water itself being subject to an established regimen.

(*e*) If both types of meter are used simultaneously in moderately perturbed water, the disparity between the discrepant velocities thus determined by the still-water rating may be taken as a basis for correcting the discrepant velocities.

(*f*) If the discharge by the weir be considered correct, the average error of the Haskell meter in Mr. Murphy's experiments was 0.02% on the side of deficiency, with a probability of its being slightly larger, and the average error of the cup meters was in the neighborhood of 1½% on the side of excessive discharge.

(*g*) It would seem to follow that current-meter observations based on still-water ratings, without further correction, should be made with great caution. On the other hand, it seems certain that the correction for a cup meter when run at a good meter station on an open river is not large, and the corresponding correction for a screw meter may be negligible.

6.—It cannot be said that the Pitot tube used at Massena was an unrated instrument. It is an exact copy of one which was investigated as "Tube *N*" by Mr. White in the paper already referred to. It has a coefficient equal to unity.

In speaking of the accuracy of Pitot tubes, two distinct reactions must be kept in mind. One is the impulse of the moving water on the dynamic orifice, the other is the reaction of the static pressure

* At the bottom of page 374 of Mr. Murphy's paper it is stated that there was a slight agitation of the water during the experiments.

Mr. Groat. and the moving water on the static orifice. If the static orifice is properly designed, the motion of the water, or the dynamic reaction, will have no effect on it; but the motion of the water will exert either suction or a positive pressure on an improperly designed static opening. It is this latter difficulty which has caused most of the uncertainty as to Pitot tubes.

Mr. White's experiments showed, in the most convincing manner, that the coefficients of all dynamic orifices of circular section, be they cylindrical, diverging, or converging, are exactly equal to unity. This was accomplished by towing a boat through the water with a number of different types of circular dynamic orifices attached somewhat in advance of the stem. The water stood at exactly the same level in all the risers, and the total head raised was equal to the sum of the velocity head and static head in each case.

7.—Although the paper does not essay to offer at once a complete remedy for the erratic behavior of current meters in perturbed water, it does point out the interesting fact that a close estimate of the true velocity may be made by using a screw meter of good design. If still greater accuracy is desired, both cup and screw meters may be used, the records being corrected by the observed relative deviations produced in some such manner as the rocking tests described briefly in the paper; or, even better still, the screw meter may be rated by a properly constructed and properly manipulated Pitot tube, under the exact conditions and at the exact point, or points, where the meter is to be used.

8.—The writer has oscillated a number of meters of different types, both longitudinally (up and down stream) and transversely (athwart stream and vertically). In all cases the cups were affected by increments several times as large as the decrements of the screws. This was for violent, as well as moderate, vibrations. It is true that the ratio is a variable, values ranging from 3 to 6 having been noted; but, when the error between the cup and screw is not more than 7%, the application of the extreme ratios leads to corrections for the screw meter of only $1\frac{3}{4}$ and 1 per cent. Thus, an error of 100% in the true value of the ratio makes a difference in the final discharge of only $\frac{3}{4}$% for the case cited.

This method, of course, is only a practical way of getting an approximate result, but the approximation is shown to be close.

9.—A current meter which cannot be rated with sufficient accuracy from a boat will show even greater discrepancies when run at the average gauging station. The United States Lake Survey has done a great work on the Niagara and St. Lawrence Rivers, and its ratings were frequently made from a boat. The writer knows that screw meters were used extensively on this important work, but does not know whether cup meters were also used. If the boat ratings had

been unsatisfactory, it is certain that that fact would have led to a condemnation of such ratings. Possibly, if cup meters had been used, the boat ratings would have proven unsatisfactory. Of course, a rating made under laboratory conditions will plot almost exactly on a smooth curve; but a rating of a screw meter of good design will be nearly as good, if made from a boat. Mr. Groat.

It is presumed by the writer that the changes in the method of rating referred to by Mr. Hoyt were such as might be made by placing the weight too close to the wheel, or by running it near the sides, or bottom, of the tank. These are abnormal ratings, of a character which does not affect the question at hand.

10.—It has already been explained that the large Price meter was selected for the tests at Massena for the sake of comparison with the results of a former test in the same tail-races with a meter of this particular type. The writer is of the opinion that the results would have been substantially the same, even if the small Price meter had been substituted for the larger pattern. It must be emphasized that the conditions of discharge for much the greater portion of the gauging section at Massena were as good as those most frequently encountered at a good gauging station. The reliability of these tests cannot be questioned on this account.

The cup meter ran with great fidelity at any given point in the race. Its rating there, relatively to that of the Haskell meter or Pitot tube, was perfectly definite, a fact which does not seem to have been known heretofore. The trouble was not with the cup meter, but with the application of the still-water rating to the case. A raceway of uniform section can certainly be gauged within 1% of the true discharge. This statement is made because any given condition of discharge can be exactly reproduced at the observer's pleasure. This is one of the facts demonstrated at Massena.

11.—The distribution factor used in the statistical method is essentially a form factor. Its value depends on the shape of the solid constructed out of the components of velocity which are normal to the cross-section when these components are erected at their points of application. To every shape a particular value of the distribution factor attaches, but it is not true, conversely, that to every value of the distribution factor a particular shape must attach. It is only necessary that shapes be equivalent as to discharge in order that two or more shapes may have the same value for their distribution factors. Hence it is not at all improbable that such changes of distribution actually take place at gauging sections without changing the discharge curve or rating table.

The mean velocity in the cross-section is the quotient obtained by dividing the total discharge by the area of the cross-section. The statistical method relies on this definition. In the turbine tests at

Mr. Groat. Massena, the value of the distribution factor was ascertained by drawing the familiar contours of equal velocity over the cross-section of the race. The volume under these contours was computed, and, when expressed in proper units, was numerically equal to the discharge. This discharge was then divided by the area in order to ascertain the mean velocity.

By definition, the distribution factor is the ratio of the above determined mean velocity to the sum of the velocities recorded at the 42 meter points in the race. By actually computing this ratio, in the foregoing manner, for several of the turbine tests where the conditions were substantially the same, it was found that the value of the ratio—that is, of the distribution factor—was constant for given conditions. Consequently, it follows that the mean velocity in the tailrace may be ascertained simply by multiplying the sum of the observed velocities by the proper value of the distribution factor.

Though a somewhat different procedure was adopted in the Southern work, it was clearly proven that the distribution factor is a definite function of the gauge height, or stage of the river, which, when once reduced to a curve, may thereafter be used as the quickest possible means of ascertaining the mean velocity in the gauging section. This is accomplished simply by multiplying the sum of the velocities at a given set of meter points by the corresponding value of the distribution factor, as read from its locus.

In short, it has been demonstrated by these experiments that, in current-meter work, and discharge gauging in general, one may deal with aggregates of velocity area, and discharge, instead of individual values, with a gain in accuracy, time, labor, and satisfaction, to be realized in no other way.

Take, for example, the case of an observer who has just hurried to a gauging station to measure a discharge at a particular stage of the stream. He drops his meter at two or three dozen permanently established meter points (supposing the river is, say, 250 ft. wide), adds up all his velocities, compares the sum with his statistical velocity curve, finds the sum correct, and then multiplies it by a factor taken from a curve, the product being the discharge. The whole work may be done within an hour or two. Moreover, the observer knows his work is correct, because his velocity observations are checked.

12.—It is very certain that the Geological Survey has done excellent work in studying velocity curves and in formulating therefrom a few simple laws which govern the discharge of rivers. Much of this work is due to Mr. Hoyt with whose name is linked some of the most valuable hydraulic investigations.

For the most part, the writer's studies have been connected with shallow turbulent rivers, as he is frequently compelled to examine streams of this class with regard to the character of their flow. It

may be of interest to know that he has arrived at two conclusions connected therewith. They are: Mr. Groat.

(*a*) A shallow turbulent stream may be gauged satisfactorily with a screw meter, or, better still, with a cup and a screw meter simultaneously.

(*b*) The proper point in a shallow turbulent section at which to take the meter readings is at mid-depth, otherwise the meter is too near the surface or too near the bottom to admit of accuracy.

Like Mr. Hoyt, the writer has summed up the measured discharges of two or more tributaries to find a good check on the discharge of the main stream. This does not prove, however, as a necessary conclusion, that the work at any of the stations is correct, as all may be, and sometimes are, affected by the same constant error.

In reply to Mr. Staniford: The writer has had only a limited experience with the current meter in salt water, in the form of a ship's log. This instrument was of the screw type, the runner being very similar to that of the Haskell meter. With care, such an instrument will give distances, and, therefore, velocities, well within 2%, even when the sea is agitated, assuming, of course, that there is no current or surface drift.

The Haskell meter was designed by Dean Haskell when he was working in salt water, and is adapted for use in both salt and fresh water. These meters have been used in coast harbors by officers of the U. S. Engineer Corps, and the Coast and Geodetic Survey; these officers would undoubtedly be able to state how fresh- and salt-water ratings compare.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

EXPERIMENTS ON WEIR DISCHARGE.

Discussion.*

By C. H. Pierce, Assoc. M. Am. Soc. C. E.

C. H. Pierce, Assoc. M. Am. Soc. C. E. (by letter).—Certain comparisons of current-meter measurements with weir discharge by standard formulas, made by the writer and other members of the United States Geological Survey in Hawaii during 1911, verify the conclusions reached by the authors in their experiments on weir discharge. Mr. Pierce.

The weirs were on irrigation ditches which offered excellent opportunities for velocity-area measurements, either in flumes or prismatic sections, within short distances of the weirs. The methods of making the velocity-area determinations were those regularly used by the engineers of the Geological Survey, as follows: The total cross-section having been divided into a number of section units, the mean velocity and area of each were measured; the product of the mean velocity and area of each individual section gave the discharge for that section, the total discharge being the sum of those of the individual sections. In most cases the sections were ½ ft. wide, and they always extended vertically from the water surface to the bottom of the channel. The mean velocity of the section was taken as one-half the sum of the means for the verticals on each edge of the section, the mean velocity for the vertical being one-half the sum of the velocities at two-tenths and eight-tenths of the depth. In this way, four determinations enter into the computation of velocity for any individual section.

The velocity measurements were made with small Price electric current meters which had been rated by the United States Bureau of Standards at Chevy Chase Lake, Maryland.

* This discussion (of the paper by W. G. Steward, Esq., and J. S. Longwell, Jun. Am. Soc. C. E., published in February, 1913, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Pierce.

Meter discharge comparisons were made with a large number of weirs, but the measurements in all cases did not cover a sufficient range in stage for a complete rating. The results of comparisons with four typical weirs will be given. These weirs were all much larger than those used by the authors, and may be considered as types of the better class, being well constructed, and designed to meet the requirements of standard weir formulas. Two of the weirs were rectangular; one had a 10-ft. crest and the other nine panels, each 29 in. wide. The other two weirs were of the Cippoletti type, having crest widths of 17 and 25 ft., respectively.

The comparisons covered a range in stage equal to that reached by the weirs under ordinary working conditions.

10-Foot Rectangular Weir.—This was an especially well-constructed weir, with a long approach channel and good end contractions. Its crest and sides were formed by a sharp-edged steel plate set in concrete. The width of the approach channel at the crest level was 23 ft.; the depth below crest level was 5.3 ft., with side slopes of 1 to 4, giving an approach channel having an area of 115 sq. ft. below crest level. A continuous record of the head on the crest was obtained by an automatic clock register.

Six discharge measurements, taken a short distance below the weir, give a rating curve which is very well defined between heads of 0.4 and 1.6 ft.

A comparison of the discharge curves obtained from the Francis formula,

$$Q = 3.33\,(b - 0.2h)\,h^{\frac{3}{2}}$$

and from current-meter measurements, is shown by Table 19 and by Fig. 5.

Nine-Panel Rectangular Weir.—Each of the nine panels in this weir had a crest width of 29 in. The crests and sides of the panels were formed by sharp-edged steel plates, but it is probable that, at the higher stages, the end contractions were not complete. The pool was approximately circular, and the weir panels were placed on a chord; water was delivered to the pool by a timber flume, 8 ft. wide and 6 ft. deep, the direction of flow of the water on entering the pool from the flume being nearly at right angles to the line of the weir. Here, also, a continuous record of the head on the crest was obtained with an automatic clock register.

Twelve discharge measurements in the flume above the pool give a rating curve which is very well defined between heads of 0.4 and 1.6 ft.

A comparison of the discharge curves obtained from the Francis formula and from current-meter measurements, is shown by Table 19 and by Fig. 5.

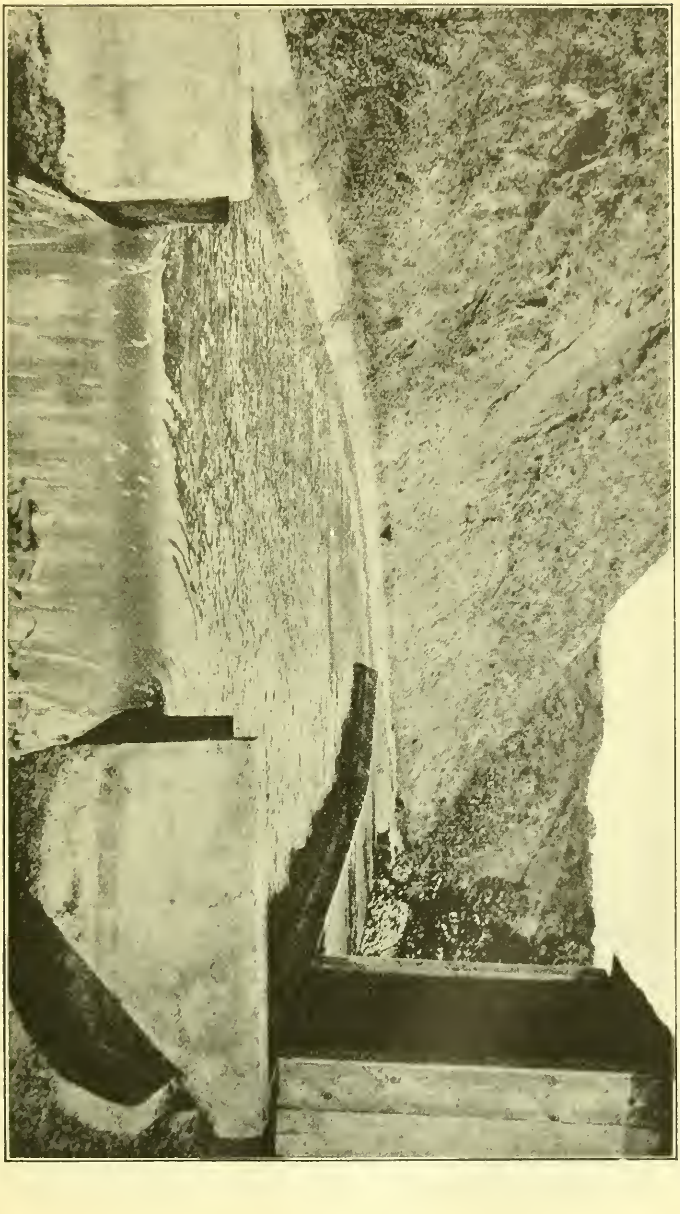

FIG. 4.—TEN-FOOT RECTANGULAR WEIR.

Mr. Pierce.

TABLE 19.—COMPARISON OF DISCHARGE BY WEIRS AND BY VELOCITY-AREA MEASUREMENTS.

Average gauge height, in feet.	Variation in gauge height, in feet.	Discharge by Francis formula, in second-feet.	Discharge by current meter, in second-feet.	Difference, in second-feet.	Percentage of difference.
10-FOOT RECTANGULAR WEIR.					
0.471	0.001	10.7	11.7	1.0	9.3
0.716	0.000	19.9	21.5	1.6	8.0
0.984	0.000	31.7	35.0	3.3	10.4
1.156	0.000	40.4	43.0	2.6	6.4
1.282	0.004	47.1	50.6	3.5	7.4
1.474	0.000	57.9	62.7	4.8	8.3
9-PANEL RECTANGULAR WEIR.					
0.45	.....	21.0	23.6	2.6	12.4
0.54	.....	27.4	30.0	2.6	9.5
0.67	.....	37.6	42.9	5.3	14.1
0.67	.....	37.6	43.8	6.2	16.5
0.68	.....	38.4	43.4	5.0	13.0
0.84	.....	51.8	60.7	8.9	17.2
0.93	.....	60.0	69.3	9.3	15.5
0.94	.....	60.8	70.0	9.2	15.3
0.97	.....	63.6	74.9	11.3	17.8
1.22	.....	87.8	103.	15.2	17.3
1.47	.....	113.	138.	25.	22.1
1.47	.....	113.	143.	30.	26.5
25-FOOT CIPPOLETTI WEIR.					
0.405	0.010	21.4	20.9	− 0.5	− 2.3
0.662	0.007	44.8	46.3	+ 1.5	+ 3.3
0.845	0.005	64.7	70.8	+ 6.1	+ 8.6
0.852	0.005	65.5	71.8	+ 6.3	+ 8.8
17-FOOT CIPPOLETTI WEIR.					
0.37	0.00	12.7	12.3	− 0.4	− 2.2
0.51	0.00	20.6	19.5	− 1.1	− 5.3
0.62	0.00	27.6	26.9	− 0.7	− 2.5
1.12	0.02	67.1	68.5	+ 1.4	+ 2.1

25-Foot Cippoletti Weir.—This weir had a sharp-edged crest and sides, formed by a steel plate set in concrete. The crest width was 25 ft. and the end inclinations were 1 to 4. Four current-meter measurements, taken in a lined section of the ditch a short distance above the weir, define a rating curve between heads of 0.4 and 0.9 ft. An automatic clock register gave a continuous record of the head on the weir.

A comparison of the discharge curves obtained from the Francis formula and from current-meter measurements is shown by Table 19 and by Fig. 6. The curves cross at about 0.55 ft. head, showing that, for the lower heads on this weir, the end inclinations should slightly exceed 1 to 4, in order to provide complete compensation for end con-

Mr. Pierce.

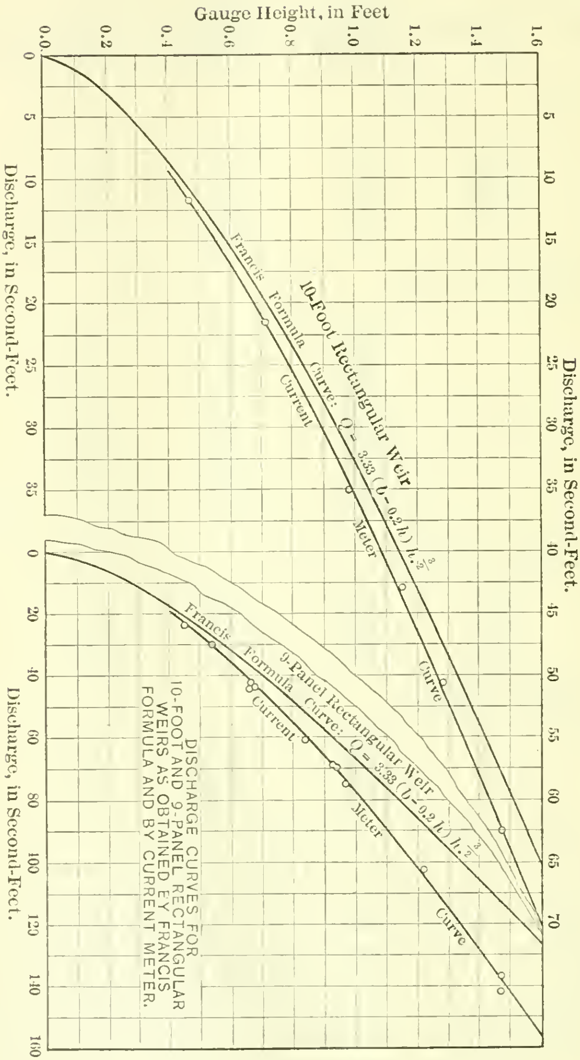

Fig. 5.

tractions.* At the higher stages there was some velocity of approach at the center of the weir, diminishing to an inappreciable amount at the sides. Mr. Pierce.

17-Foot Cippoletti Weir.—This was similar in construction to the 25-ft. weir, and likewise had a sharp-edged crest and sides, formed by a steel plate set in concrete. The crest width was 17 ft. and the end inclinations were 1 to 4. Four current-meter measurements, taken in an unlined section of the ditch about $\frac{1}{4}$ mile above the weir, define a rating curve between heads of 0.3 and 1.2 ft. An automatic clock register provided a continuous record of the head on the weir.

A comparison of the discharge curves obtained from the Francis formula and from current-meter measurements is shown by Table 19 and by Fig. 6. The curves cross at about 0.8 ft., indicating that, for this weir, the end inclinations should somewhat exceed 1 to 4 for heads less than 0.8 ft., if the effect of end contraction is to be completely compensated. There was no appreciable velocity of approach at any stage.

General Conclusions.—The results of these comparisons agree with those obtained by the authors as far as showing that, for rectangular weirs, the discharge, as computed by the Francis formula,

$$Q = 3.33\ (b - 0.2h)\ h^{\frac{3}{2}}.$$

is less than the actual discharge. This is found to be true for all stages covered by the comparisons, the percentage of difference varying, but usually being greater for the lower heads, except for weirs which were affected by velocity of approach.

The comparisons made by the velocity-area method seem to give a smaller departure from the Francis formula than those made volumetrically by the authors. It must be remembered, however, that the weirs were much larger than those used in the Boise experiments, and that the heads were relatively less, being well within the limits for which weir formulas are considered applicable; also, that the weirs were in actual operation and adjusted to every-day conditions. It appears that the authors extended their experiments on the small 6-in. and 12-in. weirs to heads much greater than should be allowed for those sizes.

For the 17-ft. and 25-ft. Cippoletti weirs at low heads, the Francis formula agrees fairly well with the results obtained by the velocity-area method, the formula giving a slightly greater discharge. For a discharge of more than about 40 sec-ft., in each case, the curves diverge quite rapidly, with the velocity-area discharge the greater.

As a result of comparative measurements by the velocity-area method with several types of weirs varying in size from 12 in. to 30 ft., the

* Water-Supply Paper No. 200, U. S. Geological Survey, p. 48.

Mr. Pierce.

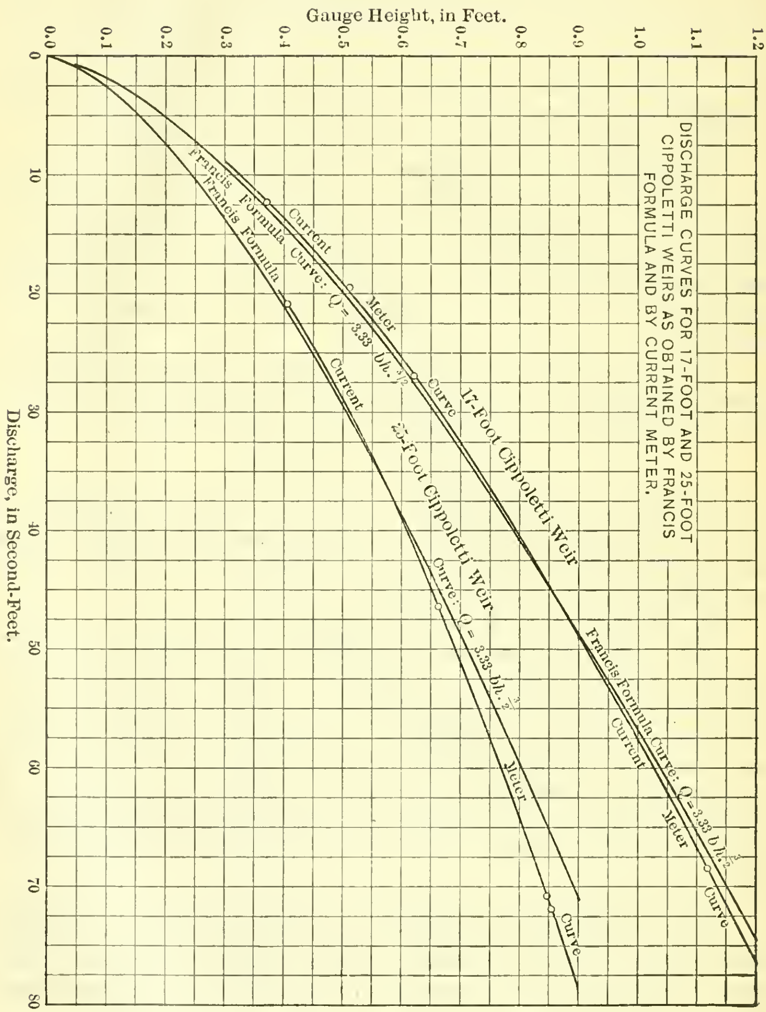

Fig. 6.

writer has reached the conclusion that the design and construction of the weir and the method of delivering the water to the crest greatly modify the theoretical discharge, and that no coefficient can be made to apply accurately to different weirs, although of the same type, if constructed under different designs. What is needed is a standard form of construction and a rating for each type of weir. A certain type should be designed, say, for delivering not more than 1 sec-ft. of water, and a rating should be provided for this weir, with definite limits as to operating head; a second type should be designed for quantities between 1 and 5 sec-ft., with a corresponding rating; and, in like manner, weirs of different capacities should be designed for specified limits of head. The rating should be determined experimentally for each type of weir, and from direct observations on that particular type. A set of standard plans and specifications for every rating would make it possible for different persons to construct weirs, and under different conditions, but according to the same design, and giving comparable results. The rating for any weir might be expressed algebraically, but probably not in the simple exponential equation of the Francis formula. Mr. Pierce.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EBENEZER SMITH WHEELER, M. Am. Soc. C. E.*

DIED JANUARY 5TH, 1913.

Ebenezer Smith Wheeler was born at Buckingham, Wayne Co., Pa., on August 27th, 1839. In 1844, he moved with his parents to Noble County, Indiana, and it was there that he received his early training preparatory to entering the University of Michigan. He was graduated from that University in the Class of 1867, with the degree of Civil Engineer. Later, in 1897, on account of his many engineering achievements and general culture, his Alma Mater conferred on him the honorary degree of Master of Science.

Mr. Wheeler's entire engineering work was performed in the service of the United States, and mainly in connection with projects for the improvement of navigation on the Great Lakes. He has left many enduring monuments which will continue to make navigation safer and reduce the cost of transportation; these bear testimony more clearly than words as to his engineering ability. He first entered the service of the United States under the U. S. Lake Survey in 1863 as Assistant Engineer, and continued in that work until 1882. While in this position, Mr. Wheeler had much to do with the early triangulation of the Great Lakes, and to the accuracy of his work is due largely the reliability of the charts and maps used as guides by navigators. To his care was entrusted the delicate work of determining the lengths of the primary base lines in the Great Lakes triangulation system, on which all extensions of this system were founded. While engaged on this work, Mr. Wheeler devised the method of using a metal bar in melting ice as a means of standardizing steel tapes and base-measuring apparatus. This method of standardization was afterward adopted and generally used by others in this branch of engineering.

In 1882, Mr. Wheeler entered the service of the U. S. Engineer Corps under the officer stationed at Detroit, Mich. From that time until 1897, he was stationed at Sault Ste. Marie, Mich., in immediate charge of all Government work in that vicinity. Under his direct supervision were constructed the Poe Lock, the widening and deepening of the St. Marys Falls Canal, the Hay Lake Channel, the 20 to 21-ft. channel through St. Marys River, and many minor improvements to navigation.

* Memoir prepared by Charles Y. Dixon, M. Am. Soc. C. E.

In 1897, Mr. Wheeler was designated by the authorities at Washington to supervise the making of a preliminary survey for the Nicaragua Canal project in Central America. During 1897, he had direct charge of all field parties in Nicaragua, and during 1898, he was in Washington preparing his report and plans for submission to the Nicaragua Canal Commission. Mr. Wheeler's report was so complete that there was no doubt that the Nicaragua Canal was a feasible engineering project, but this route was afterward abandoned for the Panama Canal. While engaged on his work in Nicaragua, Mr. Wheeler contracted a fever from the effects of which he never fully recovered, although, up to that time, his had been a most rigorous constitution.

On his return from work with the Nicaragua Canal Commission, in 1898, Mr. Wheeler was assigned to duty in the U. S. Engineer Office at Detroit, Mich., as Chief Assistant Engineer, and he continued in that position until his death. During this period, he had direct charge of the construction of the new St. Clair Flats Canal and of the breakwater at Mackinac Island. His engineering experience was also found to be invaluable when acting in an advisory way in connection with all other important works of the District.

While stationed at Detroit, Mr. Wheeler found time to make many investigations of value along lines indirectly connected with his work. The most important of these led to his invention of the Wheeler bathometer. This device has been placed on a number of freight vessels on the Great Lakes to give warning when in shoal water. It has been operated successfully for several seasons, and is destined to be the means of saving life and property on the Great Lakes.

A mere recital of the works with which Mr. Wheeler was connected is ample evidence of his engineering ability; but he was much more than an able engineer; he was well informed on many subjects, about which knowledge would be expected to be obtained by technical training only. He was, withal, a modest, unobtrusive, cultured gentleman. He was gifted with a keen sense of humor, and had that rare and happy faculty which enabled him to solve the difficulties of others and to illustrate his own thoughts with a good story well told. He was a most delightful companion, whom it was a genuine privilege to have known, and no one could know him well without being the better for it.

In 1874, Mr. Wheeler married Miss Clara P. Fuller who, with one daughter and one son, survives him. To his family the sympathy of a host of friends is extended, for truly all who knew him were his friends.

He was a member of many technical and fraternal societies, including the Michigan Engineering Society, the Detroit Engineering Society, the Detroit University Club, and the Masonic fraternity.

Mr. Wheeler was elected a Member of the American Society of Civil Engineers on November 7th, 1883.

PAPERS IN THIS NUMBER

"STATICAL LIMITATIONS UPON THE STEEL REQUIREMENT IN REINFORCED CONCRETE FLAT SLAB FLOORS." JOHN R. NICHOLS. (To be presented May 21st, 1913.)

"RECENT IMPROVEMENTS IN LEVELING INSTRUMENTS." DUNBAR D. SCOTT. (To be presented May 21st, 1913.)

"TIDAL PHENOMENA IN THE HARBOR OF NEW YORK." H. DE B. PARSONS. (To be presented May 7th, 1913.)

"THE PHILOSOPHY OF ENGINEERING." MAURICE G. PARSONS. (To be presented June 4th, 1913.)

PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

PAPERS IN THIS NUMBER

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William P. Morse

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXIX—No. 3

March, 1913

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXIX—No. 3

MARCH, 1913

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1913

American Society of Civil Engineers

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To Codify Present Practice on the Bearing Value of Soils for Foundations, etc.: Robert A. Cummings, Edward C. Shankland, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner, Frank M. Kerr.

The House of the Society is open from 9 a. m. to 10 p. m. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

House of the Society—220 West Fifty-seventh Street, New York.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

February 19th, 1913.—The meeting was called to order at 8.30 P. M.; Vice-President J. Waldo Smith in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 131 members and 19 guests.

A paper by John N. Brooks, Jun. Am. Soc. C. E., entitled "The Infiltration of Ground-Water into Sewers," was presented by the author, and the subject was discussed by Messrs. John H. Gregory, G. L. Christian, Edward S. Rankin, E. Kuichling, Charles Saville, Kenneth Allen, and the author.

A paper by J. L. Meem, Assoc. M. Am. Soc. C. E., entitled "A Suggested Improvement in Building Water-Bound Macadam Roads," was presented by J. C. Meem, M. Am. Soc. C. E., and illustrated with lantern slides. The paper was discussed by Messrs. A. H. Blanchard, who illustrated his remarks with lantern slides, James Owen, and William J. Boucher, and communications on the subject from Messrs. F. G. Frink, C. H. Sweetser, and W. W. Crosby, were read by the Secretary.

A paper entitled "On Long-Time Tests of Portland Cement" by I. Hiroi, M. Am. Soc. C. E., was presented by the Secretary. Written discussions from Messrs. Chandler Davis, L. J. Le Conte, and George G. Honness were presented by title on account of the lateness of the hour.

The Secretary announced the following deaths:

John Fritz, elected Member, July 5th, 1893; Honorary Member, September 5th, 1899; died February 12th, 1913.

David McNeely Stauffer, elected Member, September 2d, 1874; died February 5th, 1913.

Adjourned.

March 5th, 1913.—The meeting was called to order at 8.30 p. m.; Vice-President J. Waldo Smith in the chair; Charles Warren Hunt, Secretary; and present, also, 121 members and 12 guests.

The minutes of the Annual Meeting, January 15th, and of the meeting of February 5th, 1913, were approved as printed in *Proceedings* for February, 1913.

A paper by Caleb Mills Saville, M. Am. Soc. C. E., entitled "Hydrology of the Panama Canal," was presented by title. The Secretary presented a communication on the subject from Gen. Henry L. Abbot, and the paper was discussed by T. Kennard Thomson, M. Am. Soc. C. E.

At the request of the Chairman, Robert Ridgway, M. Am. Soc. C. E., addressed the meeting on the subject of the Dual System of Rapid Transit in New York City, and the matter was discussed by Messrs. W. J. Boucher, N. P. Lewis, L. D. Rights, and others.

Also at the request of the Chairman, Thomas H. Wiggin, M. Am. Soc. C. E., described a design for a large cast-steel cover in a shaft on the tunnel for the Catskill Aqueduct, and the subject was discussed by Messrs. H. F. Dunham, F. W. Skinner, V. H. Hewes, W. B. Bamford, and others.

The Chair having asked for discussion on the question of making the meetings of the Society more interesting, Messrs. T. Kennard Thomson, W. J. Boucher, and the Secretary expressed their views.

The Secretary announced the election of the following candidates on March 4th, 1913:

As Members

William Goodson Ames, Havana, Cuba
Herman Peter Andresen, Chicago, Ill.
John Toner Barr, Pittsburgh, Pa.
William Godley Coughlin, Renovo, Pa.
William Reid Felton, Lonetree, Mont.
Robert Richard Gales, Bengal, India

Friederich Ernst Giesecke, College Station, Tex.
John Charles William Greth, Pittsburgh, Pa.
Edward Sherman Jackson, Gooding, Idaho
Charles Nicholas Monsarrat, Montreal, Que., Canada
William Edward Parker, Washington, D. C.
Hugh Pattison, Chicago, Ill.
William Powell Rothrock, Pittsburgh, Pa.
Joseph Aloysius Rourke, Boston, Mass.
Charles George Schade, Canonsburg, Pa.
Francis Lee Weakland, New York City

As Associate Members

William Thomas Adelhelm, Philadelphia, Pa.
Robert Burns Haldane Begg, Lawrence, Kans.
Frank Charles Boes, West New Brighton, N. Y.
Richard Bailey Cook, Easton, Pa.
Ray Edgar Fulcher, Burbank, Wash.
William Goldsmith, New York City
Harry Matt Gray, Poughkeepsie, N. Y.
James Edward Grimes, Chicago Heights, Ill.
John Wardwell Howard, Boston, Mass.
Charles Brown Kingsley, Toronto, Ont., Canada
Ernest Eugene Lee, Culebra, Canal Zone, Panama
Hal Helm Logan, Denver, Colo.
Stanley Macomber, Centralia, Wash.
Joseph Newall McKernan, Plainville, Conn.
John Montgomery Mahon, Jr., Harrisburg, Pa.
Paul McClary Paine, Fellows, Cal.
Arturo Pani, City of Mexico, Mexico
Abraham John Ruth, Cananea, Sonora, Mexico
James Cummin Stevenson, San Antonio, Tex.
Harris Martyn Stringfellow, City of Mexico, Mexico
George Roscoe Blaine Symonds, Manila, Philippine Islands
James Olin Wanzer, Los Baños, Cal.
Herbert Cassidy Wells, Washington, D. C.

As Juniors

Andrew Lewis Ackhart, Toronto, Ont., Canada
Harry Ross Daubenspeck, Johnson, N. C.
William Fisher Drury, Orono, Me.
Richard Henderson Eurich, Montclair, N. J.
Robert Nesbitt Greene, Southington, Conn.
James Warren Ingalls, Lynn, Mass.
Eugene Frederick Mail, Robinson, Ill.

SEATON SCHROEDER, JR., Altmar, N. Y.
ROBERT L'HOMMEDIEU TATE, Princeton Mills, W. Va.
THEODORE ERNST VELTFORT, Keokuk, Iowa
CHAUNCY WERNECKE, Seattle, Wash.

The Secretary announced the transfer of the following candidates on March 4th, 1913:

FROM ASSOCIATE MEMBER TO MEMBER

CLIFFORD GEORGE DUNNELLS, Pittsburgh, Pa.
RANDALL DUNBAR GARDNER, Boston, Mass.
KENNETH CROTHERS GRANT, Pittsburgh, Pa.
GUY WALTER HARRIS, Los Angeles, Cal.
ERNEST EMMANUEL HOWARD, Kansas City, Mo.
ROBERT PARSONS HOWELL, Phillipsburg, N. J.
ARTHUR ERNEST MORGAN, Memphis, Tenn.
JOHN RICHARD PILL, Corona, Ala.
BLAIR RIPLEY, Toronto, Ont., Canada

FROM JUNIOR TO ASSOCIATE MEMBER

HAROLD PHILLIPS FARRINGTON, New York City
WILLIS GEORGE FROST, Healdsburg, Cal.
SAMUEL ARNOLD GREELEY, Winnetka, Ill.
WILLIAM HEYMAN, Jersey City, N. J.
HENRY JAMES KESNER, Berkeley, Cal.
THOMAS GEORGE MCCRORY, Seattle, Wash.
RICHARD ELAM MIETH, Portland, Ore.
DAY IRA OKES, Minneapolis, Minn.
WALTER MAX SANGER, Toledo, Ohio
OLIVER EARLE YOUNG, Chicora, Fla.

The Secretary announced the following deaths:

Sir WILLIAM HENRY WHITE, elected Honorary Member, December 16th, 1904; died February 27th, 1913.

ALBERT SAFFORD CHEEVER, elected Member, June 7th, 1893; died February 17th, 1913.

RUDOLPH FINK, elected Member, September 21st, 1870; died February 1st, 1913.

FRANK SOULÉ, elected Member, March 1st, 1905; died February 14th, 1913.

ARTHUR GARFIELD CRYSLER, elected Associate Member, August 31st, 1909; died October 22d, 1912.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

April 2d, 1913.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "Shearing Strength of Construction Joints in Stems of Reinforced Concrete T-Beams, as Shown by Tests," by Lewis J. Johnson, M. Am. Soc. C. E., and John R. Nichols, Jun. Am. Soc. C. E.; and "Fremantle Graving Dock: Steel Dam Construction for North Wall," by Joshua Fielden Ramsbotham, Assoc. M. Am. Soc. C. E.

These papers were printed in *Proceedings* for February, 1913.

April 16th, 1913.—8.30 P. M.—At this meeting two papers will be presented for discussion as follows: "Kinetic Effect of Crowds," by C. J. Tilden, Assoc. M. Am. Soc. C. E.; and "Some Experiments with Mortars and Concretes Mixed with Asphaltic Oils," by Messrs. Arthur Taylor and Thomas Sanborn.

These papers are printed in this number of *Proceedings*.

May 7th, 1913.—8.30 P. M.—A regular business meeting will be held, and a paper by George Schobinger, Jun. Am. Soc. C. E., entitled "Colorado River Siphon," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-fifth Annual Convention will be held at Ottawa, Ont., Canada, some time between June 1st and July 15th, 1913, the precise date having not yet been fixed.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April,

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Gavin N. Houston, M. Am. Soc. C. E., 409 Equitable Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, and, until further notice, will take place at the Colorado Traffic Club.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Meeting)

February 8th, 1913.—The meeting was called to order; President Ketchum in the chair; G. N. Houston, Secretary; and present, also, 16 members and 3 guests.

A Progress Report of the Legislative Committee was read by J. E. Field, M. Am. Soc. C. E., and approved.

The Secretary read the Report of the Auditing Committee, which was approved and the Committee discharged.

After discussion on the proposed Amendment to the By-Laws of the Association concerning the election of officers, and on motion, duly seconded, it was ordered that the Nominating Committee be increased to five members to be selected at large from the Association by the Chair.

A paper on "Surety Bonds," by C. L. Martindale, Resident Vice-President of the Equitable Surety Company, was read by Vernon L. Foxwell, Manager of the American Bonding Company, who also discussed the subject. The paper was discussed from the legal standpoint by H. H. Clark, Attorney for the U. S. Fidelity and Guarantee Company.

Adjourned.

Atlanta Association

On March 14th, 1912, the Atlanta Association of Members of the American Society of Civil Engineers was organized, with the following officers: Arthur Pew, President; William A. Hansell, Jr., Secretary; and Messrs. James N. Hazlehurst and Alexander Bonnyman, Members of the Executive Committee. The Association will hold its meetings in the house of the University Club.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 302 Board of Trade Building, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From February 3d to March 4th, 1913)

DONATIONS.*

ROADS, PATHS AND BRIDGES.

By Logan Waller Page, M. Am. Soc. C. E. (The Farmer's Practical Library.) Cloth, 7½ x 5 in., illus., 263 pp. New York, Sturgis & Walton Company, 1912. 75 cents.

The Introduction states that it is the purpose in this book to give, in a concise and elementary form, the fundamental principles governing the construction of roads, paths and bridges for farm and neighborhood purposes, and to present the details of construction and maintenance in connection therewith so that they may be easily followed. The author has also included matter relating to the origin and development of road building and the progress of road legislation and administration, as well as a few of the economic phases of the road question. The Chapter headings are: History of Road Building; Road Legislation and Administration; Locations, Surveys, Plans, Specifications; The Earth Road; The Sand-Clay Road; The Gravel Road; The Broken Stone Road; The Selection of Materials for Macadam Roads; Maintenance and Repairs; Roadside Treatment; Modern Road Problems; Paths; Culverts and Bridges; Bibliography; Index.

HANDBOOK OF RAILROAD EXPENSES.

By J. Shirley Eaton. Morocco, 7½ x 5 in., 12 + 559 pp. New York and London, McGraw-Hill Book Company, 1913. $3.00.

The present form of accounting and accounting statistics required of the railroads by the Interstate Commerce Commission is, the preface states, the product of many years of experience and careful study. The author states that as the questions involved in the transportation field, individual and social, will some day be discussed in a manner which will deserve to be termed "scientific", he has attempted to comprehend in these classifications all the economic forces which have play in the transportation business. His intention has been to present a handbook containing a "Classification of Railroad Expenses" which is reasonably complete for all the purposes of the operating official or for the railroad statistician and financier. This analysis, it is said, is intended to supersede that contained in the author's "Railroad Operations—How to Know Them," published about twelve years ago. All existing indexes of expenses have been freely used in compiling this Handbook, and abridged versions of some of the texts of the Interstate Commerce Commission are included. The "Classification" of the Interstate Commerce Commission is given in full with the Commission's later corrections incorporated in the text. The Contents are: The Capital and Income Accounts; Maintenance of Way and Structures; Maintenance of Equipment; Traffic Expenses; Transportation; General Expenses; Outside Operations; Additions and Betterments; Interstate Commerce Commission Texts—Reprinted and Abridged; Index.

THE MODERN GASOLINE AUTOMOBILE:

Its Design, Construction, Maintenance and Repair. By Victor W. Pagé. Cloth, 8¾ x 5¾ in., illus., 693 pp. New York, The Norman W. Henley Publishing Company, 1913. $2.50.

In a secondary title, it is stated, that this book is a comprehensive treatise in which all the principles pertaining to gasoline automobiles and their component parts, are defined, every phase of the subject being treated in a practical, non-technical manner. After explaining the basic principles of the gasoline motor car, the author describes actual forms and mechanisms, in order, it is stated, that their practical application and the relations which the various parts bear each other may be easily understood. In addition to a thorough discussion of the automobile, its equipment, accessories, tools, supplies, spare parts, etc., detailed descriptions of the most recent innovations in 1913 models are given, such as the torpedo and other body forms, sleeve valve motors, selective sliding gearset, shaft and bevel gear drive, worm gear power transmission, magneto ignition, electric lighting systems, etc. Examples from temporary foreign and domestic practice are included, as well as about 500 illustrations which have been made from actual working drawings. The Contents are: Types of Modern Automobiles; How Power is Generated; The Principal Parts of Gasoline Engines, Their Design, Construction and Application; Constructional Details of Pistons; Liquid Fuels Used and Methods of Vaporizing to Obtain Explosive Gas;

* Unless otherwise specified, books in this list have been donated by the publishers.

Automobile Power-Plant Ignition Systems Outlined; Reason for Lubrication of Mechanism; Utility of Clutches and Gearsets Defined; The Chassis and Its Components; Wheels, Rims, and Tires; Motor Car Equipment and Accessories; Operating Advice and Explanation of Automobile Control Methods; Practical Hints to Assist in Locating Power-Plant Troubles; Keeping Up the Motor-Car Chassis; Index.

DIARY OF A ROUNDHOUSE FOREMAN.

By T. S. Reilly. Cloth, 7 x 5 in., 158 pp. New York, The Norman W. Henley Publishing Company, 1912. $1.00.

The subject-matter contained in this book first appeared anonymously in the *Railway and Engineering Review* as a serial. In order to give the man who is about to take charge of a roundhouse that grasp of administrative matters which is essential to his success, it has been assumed that a typical young machinist has been placed in charge of a roundhouse, and his experiences in that position are given. It is stated that all the characters and incidents have been taken from actual railroad life, having passed under the author's personal observation while occupying various positions as a railroad official.

ELEMENTARY PRINCIPLES OF ELECTRICITY AND MAGNETISM.

For Students in Engineering. By Robert Harbison Hough and Walter Martinus Boehm. Cloth, 7½ x 5 in., illus., 7 + 233 pp. New York, The Macmillan Company; London, Macmillan & Co., Ltd., 1913. $1.10.

The object of this book, it is stated, is to develop, in logical order, the more important numerical relations existing among the principal quantities used in electricity and magnetism, from definitions and elementary laws stated in simple language. Only those relations which are fundamental to the design of the various machines and engines used in engineering practice are developed, and no attempt has been made to include any descriptive matter. The text, it is stated, is intended to be used with lecture demonstrations involving three types of experiments, namely, those presenting elementary phenomena or independent experimental laws on which the propositions depend; verification of the principal propositions derived; and illustrations of the more important applications. Problems are included to illustrate the use of formulas, the principal equations being numbered for convenient reference. The Contents are: Magnetism; Magnetic Induction; Electrostatics; Electrostatic Induction; Capacity (Electrostatic); Electrostatic Machines; Electrodynamics (Fields); Quantity; Resistance, Difference of Potential and Capacity; Networks of Conductors; Electrodynamics (Conductors); Electro-Magnetic Induction; Magnetic Circuits; Rotating Coil in Uniform Field; Coefficients of Induction; Alternating Currents; Dynamo Electric Machinery; Appendix; Index.

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Wyoming-State Engr. 1 pam.
Wyoming-State Geologist. 1 pam.

BY PURCHASE

Tunneling: A Practical Treatise. By Charles Prelini. Sixth Edition, Revised and Enlarged. D. Van Nostrand Co., New York, 1912.

A Treatise on the Design and Construction of Roofs. By N. Clifford Ricker. John Wiley & Sons, New York; Chapman & Hall, Limited, London, 1912.

Explosives: A Synoptic and Critical Treatment of the Literature of the Subject as Gathered from Various Sources. By H. Brunswig.

Translated and Annotated by Charles E. Munroe and Alton L. Kibler. John Wiley & Sons, New York; Chapman & Hall, Limited, London. 1912.

A Textbook on Sewage Disposal in the United Kingdom. By Henry Lemmoin-Cannon. With a Foreword by *Sir* Alexander Stenning. St. Bride's Press, Limited, London, 1912.

Water Supply and Drainage Systematised and Simplified. By C. E. Housden. Longmans, Green and Co., New York and London, 1912.

The Mechanics of the Aeroplane: A Study of the Principles of Flight. By Captain Duchene. Translated from the French by John H. Ledeboer and T. O'B. Hubbard. Longmans, Green and Co., New York and London, 1912.

Military Hygiene and Sanitation. By Charles H. Melville. Edward Arnold, London, 1912.

The Una-Flow Steam-Engine. By J. Stumpf. D. Van Nostrand Co., New York; Constable & Company, Ltd., London, 1912.

Deutscher Ausschuss für Eisenbeton. Hefte 6-22. Wilhelm Ernst & Sohn, Berlin, 1911-13.

Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, insbesondere aus den Laboratorien der technischen Hochschulen. Herausgegeben vom Verein deutscher Ingenieure. Hefte 129-130. Julius Springer, Berlin, 1912.

Revue Générale des Chemins de Fer, 7e Année, 1er Semestre, January-June, 1884. Dunod, Paris.

Poor's Manual of the Railroads of the United States, 1913. Poor's Railroad Manual Co., New York.

Zeitschrift des Vereines deutscher Ingenieure: Inhaltsverzeichnis der Jahrgänge 1904 bis 1910, Band 48 bis 54. Julius Springer, Berlin, 1911.

SUMMARY OF ACCESSIONS

(From February 3d to March 4th, 1913)

Donations (including 12 duplicates)	223
By purchase	30
Total	253

MEMBERSHIP

ADDITIONS

(From February 7th to March 6th, 1913)

MEMBERS		Date of Membership.
BETTS, EDWARD EVERETT. Cons. Engr. (Edward E. Betts Eng. Co.), 703 James Bldg., Chattanooga, Tenn.	Assoc. M. M.	April 5, 1899 Feb. 4, 1913
CUNNINGHAM, EDWARD WALTER. Asst. Bldg. Insp., Dept. of Public Safety, City of Cleveland, 11020 Detroit St., Cleveland, Ohio	Assoc. M. M.	Oct. 2, 1907 Feb. 4, 1913
DARROW, WILTON JOSEPH. Cons. Engr., 70 East 45th St., New York City	Assoc. M. M.	Nov. 1, 1905 Feb. 4, 1913
HORTENSTINE, HENRY ROBERTS. Contr. Mgr., Penn Bridge Co., Beaver Falls, Pa.	Assoc. M. M.	April 3, 1907 Feb. 4, 1913
MATHESON, ERNEST GEORGE. Supt., The Foundation Co., C. P. R. Pitt River Bridge, P. O. Box 68, Coquitlam, B. C., Canada.	Assoc. M. M.	June 1, 1904 Feb. 4, 1913
NICHOLS, WALTER SWAIN. 717 Walnut St., Philadelphia, Pa.		Feb. 4, 1913
PARMELEE, CHARLES LESTER. Cons. Engr., 30 Church St., New York City		Feb. 4, 1913
SANDS, EDWARD EMMET. Superv. Engr., Dept. of Natural Resources, C. P. Ry., Calgary, Alberta, Canada	Assoc. M. M.	Sept. 5, 1911 Feb. 4, 1913
THOMAS, WILLIAM JOHN. Chf. Engr., Geo. B. Post & Sons, 1977 Morris Ave., New York City		Feb. 4, 1913
WEBBER, CHARLES PERKINS. Ingeniero Principal, Departmento de Construccion, Ferro Carriles Nationales de Mexico, Apartado 11, Tierra Blanca, Ver., Mexico	Assoc. M. M.	Oct. 7, 1908 Jan. 7, 1913
WEIR, MAX WAKEMAN. Engr., The Comm. for the Drainage of the Tonawanda and Oak Orchard Swamps, Elba, N. Y.		Feb. 4, 1913
ASSOCIATE MEMBERS		
BALDRIDGE, JAMES RAMSEY. Contr. Engr., Hennebique Constr. Co., 1170 Broadway, New York City	Jun. Assoc. M.	Dec. 4, 1906 Oct. 29, 1912
CONKLIN, CHARLES DENTON, JR. Cons. and Designing Structural Engr., Reed Bldg., Philadelphia (Res., Cheltenham), Pa.		Feb. 4, 1913
CONNELLY, JOSEPH AUGUSTINE ALOYSIUS. Asst. Engr., Public Service Comm., First Dist., 888 Forest Ave., New York City		Feb. 4, 1913
CONVERSE, WARREN HOOVER, JR. Designing Engr., The Converse Bridge Co., Ridgedale Sta., Chattanooga, Tenn.		Jan. 7, 1913

ASSOCIATE MEMBERS (*Continued*)		Date of Membership.
COOPER, SIDNEY WOODDELL. Drainage Engr., U. S. Dept. of Agri., Roswell, N. Mex.		Jan. 7, 1913
DUFFEE, LOUIS WARREN. Chf. Engr., Constr., Meridian & Memphis R. R., Box 733, Meridian, Miss.		Feb. 4, 1913
ELLSWORTH, CLARENCE EUGENE. In Chg., U. S. Geological Survey Water Supply Investigations, Yukon-Tanana Region, Alaska, 1757 You St., Washington, D. C.		Feb. 4, 1913
ENTENMANN, PAUL MAX. Asst. Engr., Public Service Comm., First Dist., 943 East 22d St., Brooklyn, N. Y.	Jun. Assoc. M.	Mar. 1, 1910 Feb. 4, 1913
ESTEN, HOWARD FOSS. Asst. Engr., N. Y., N. H. & H. R. R., 533 Hope St., Providence, R. I.	Jun. Assoc. M.	Jan. 2, 1906 Feb. 4, 1913
FINLEY, CHARLES MACFARLANE. 208 American Blk., Sioux City, Iowa		Feb. 4, 1913
GARDNER, HENRY JAMES, JR. Prin. Asst. Engr., Ricker & Minniss, 702 Ellicott Sq., Buffalo, N. Y.	Jun. Assoc. M.	Oct. 31, 1911 Feb. 4, 1913
GRANDPRÉ, AMBROSE GOULET. Supt of Constr., Marshall & Fox, 5343 Ellis Ave., Chicago, Ill.		Dec. 3, 1912
GRIMES, ERNEST EDMUND. Care, Utah Ry., 516 Felt Bldg., Salt Lake City, Utah		Feb. 4, 1913
HATCH, FREDERICK NATHANIEL. With Westinghouse, Church, Kerr & Co., 86 Philip St., Jersey City, N. J.	Jun. Assoc. M.	June 4, 1907 Feb. 4, 1913
HICKERSON, THOMAS FELIX. Associate Prof. of Civ. Eng., Univ. of North Carolina, Chapel Hill, N. C.		Feb. 4, 1913
HULL, GORDON BURNETT GIFFORD. Asst. Supt., Boquilla Dam, for Mexican Northern Power Co., Ciudad Camargo, Chihuahua, Mexico	Jun. Assoc. M.	July 1, 1909 Feb. 4, 1913
JOHNSTONE, ALAN MOORE EDWARD. 2142 Ellis Ave., New York City		Feb. 4, 1913
KEENE, WILLIAM ARCHIBALD, JR., Chf. Engr., White River Levee Dist., Box 276, Cotton Plant, Ark.		Feb. 4, 1913
McCARTHY, DANIEL FRANCIS. Bronxville, N. Y.		Sept. 3, 1912
McSWAIN, THOMAS RUCKER. City Engr., Tulare, Cal.		Jan. 7, 1913
MAYO, GEOFFREY WAINMAN. Designing Engr., Dept. of Eng. and Public Works, Care, City Engr., Manila, Philippine Islands		Dec. 3, 1912
PACKARD, AMBROSE. Engr., Pres., and Mgr., J. S. Packard Dredging Co. and Packard Hydr. Dredging Co., 93 South Angell St., Providence, R. I.		Feb. 4, 1913
REIMANN-HANSEN, ROBERT LOUIS. Head Draftsman, B. & O. R. R., 3637 Park Heights Ave., Baltimore, Md.	Jun. Assoc. M.	Oct. 3, 1905 Feb. 4, 1913

ASSOCIATE MEMBERS (*Continued*)		Date of Membership.
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SMULSKI, EDWARD. With Sanford E. Thompson, in Chg. of Dept. of Design, 141 Milk St., Room 809, Boston. Mass..		Feb. 4, 1913
STEEP, JAMES BIGELOW. Chf. Engr., Noelke Richards Iron Works, 1732 North Illinois St., Indianapolis, Ind...		Feb. 4, 1913
TAYLOR, HENRY WILLIAM. Hydr. and San. Engr., 100 State St. (Res., 346 State St.), Albany, N. Y............		Feb. 4, 1913
TIRRELL, CHARLES EDWARDS. Chf. Engr., A. Friederich & Sons Co., 106 Mill St., Rochester, N. Y......................	Jun. Assoc. M.	Sept. 4, 1906 Feb. 4, 1913
WEBSTER, ROYAL SYLVESTER. Asst. Engr., Havana Central R. R., Arsenal, Havana, Cuba..............................	Jun. Assoc. M.	Oct. 6, 1903 Feb. 4, 1913
WILSON, JOHN JUNIOR. 330 Mapleton Ave., Boulder, Colo.		Feb. 4, 1913

JUNIORS

DODGE, FRANK EARLE. Engr. on Constr., for Knickerbocker Portland Cement Co., Hudson, N. Y............. ..	Feb. 4, 1913
LEE, CHESTER SHERMAN. Engr., M. of W., Otsego & Herkimer R. R., Cooperstown, N. Y.....................	Feb. 4, 1913
LEWIS, HAROLD MACLEAN. With George W. Fuller, 170 Broadway, New York City (Res., 1511 Albermarle Rd., Brooklyn, N. Y.)...........................	Feb. 4, 1913
ORT, ALBERT AUGUST LAMBERT. Vice-Pres. and Engr. in Chg., Pitometer Dept., The Municipal Supply Co., 601 Western Union Bldg., Chicago, Ill.............	Sept. 3, 1912
PORTER, ELMER ALFRED. Asst. Engr., U. S. Geological Survey, Salt Lake City, Utah.......................	Oct. 1, 1912
RASMUSSEN, ALVIN CHRISTIAN. Asst. Engr., Insley Mfg. Co., 2250 Central Ave., Indianapolis, Ind..........	Feb. 4, 1913
WELLS, JAMES BERTRAND. With Eng. Dept., Standard Oil Co., 365 Guinda St., Palo Alto, Cal...............	Jan. 7, 1913

CHANGES OF ADDRESS

MEMBERS

ADAMS, EDWIN GRIGGS. Care, Riter-Conley Mfg. Co., Pittsburgh, Pa.

BACKES, WILLIAM JAMES. Engr., M. of W., N. Y., N. H. & H. R. R., New Haven, Conn.

BACON, THEODORE HASTINGS. Care, John F. Stevens Constr. Co., 55 Wall St., New York City.

MEMBERS (*Continued*)

Boardman, Howard Edward. Chf. Asst. Engr., Underground Lines, Buenos Aires Western Ry., Buenos Aires, Argentine Republic.

Briggs, Josiah Ackerman. Cons. Engr., 2305 Andrews Ave., New York City.

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DEATHS

CHEEVER, ALBERT SAFFORD. Elected Member, June 7th, 1893; died February 17th, 1913.

FINK, RUDOLPH. Elected Member, September 21st, 1870; died February 1st, 1913.

FRITZ, JOHN. Elected Member, July 5th, 1893; Honorary Member, September 5th, 1899; died, February 12th, 1913.

SOULÉ, FRANK. Elected Member, March 1st, 1905; died February 14th, 1913.

STAUFFER, DAVID MCNEELY. Elected Member, September 2d, 1874; died February 5th, 1913.

WHITE, *Sir* WILLIAM HENRY. Elected Honorary Member, December 16th, 1904; died February 27th, 1913.

Total Membership of the Society, March 6th, 1913, 6 823.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(February 3d to March 4th, 1913)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Institute Indicator*, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *Engineering Record*, New York City, 10c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Elektrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 25c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.

(54) *Transactions*, Am. Soc. C. E., New York City, $1.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England, 6d.
(69) *Der Eisenbau*, Leipzig, Germany.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Mining and Engineering World*, Chicago, Ill., 10c.
(83) *Gas Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Wasbington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga., 10c.

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Bridges.

The Widening of Blackfriars Bridge.* David Anderson and Harry Cunningham. (63) Vol. 190.
The Repairs on the Menai Suspension Bridge.* Edward George Rivers and Edward Harold Morris. (63) Vol. 190.
An Artistic Stone Arch.* Henry B. Seaman. (60) Feb.
Reinforced Concrete Highway Bridges.* George H. Herrold. (Paper read before the Civil Engrs. Soc. of St. Paul.) (1) Feb.
Columbia River Bridge at Trail.* (14) Feb. 8.
Testing Rig for Full-Size Concrete Bridge Slabs.* (14) Feb. 8.
Progress in Connection with the Construction of the Quebec Bridge.* (96) Feb. 13.
A Light Reinforced-Concrete Arch, La Mescla, France.* (13) Feb. 13.
New Lift Bridge of the Oregon-Washington R. R. & Navigation Co. at Portland.* (18) Feb. 15.

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Bridges—(Continued).

Bascule Highway Bridge with Overhead Balance Beams.* (14) Feb. 15.
Replacing High Timber Trestle Without Interrupting Traffic.* (14) Feb. 15.
Method of Constructing a Three-Span Skew Arch Bridge Using Steel Centers.* (86) Feb. 19.
Floors for Movable Bridges: A Summary of Practice and Experience. H. G. Tyrrell. (86) Feb. 19.
Stresses and Moments in a Rigid Frame. (86) Feb. 19.
Concrete Bridges and Culverts.* (96) Feb. 20.
Erection Costs for a Double-Track Railway Bridge.* M. A. O. Stilson. (13) Feb. 20.
Application of Electric Motors to Bascule Lift Bridges, N. Y., N. H. & H. R. R.* (18) Feb. 22; (62) Mar. 3.
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Concrete Highway Bridge at Ansonia, Connecticut.* (14) Feb. 22.
Bridge Trusses with Uniform T-Shape Members.* (14) Feb. 22.
Piers for Drawbridge over Calumet River.* (14) Feb. 22.
Improved Drawbridge Gate Latch.* (14) Feb. 22.
The Sewickley Cantilever Bridge.* (13) Feb. 27.
The Crooked River Arch Bridge.* (15) Feb. 28.
Gilbert Avenue Viaduct in Cincinnati.* (14) Mar. 1.
Consolidation par Injection de Ciment du Viaduc des Cent Arches Situé sur la Ligne Paris-Bordeaux.* Adam. (38) Feb.
Note sur le Calcul des Ponts Suspendus Rigides à Haubans et Poutre de Rigidité.* Henry Lossier et Maurice Pernollet. (33) Serial beginning Feb. 1.
Knotenpunkte von Vierendeelträgern und Verwandte Gebilde.* M. Mecklenbeck und Ehrlich. (69) Serial beginning Jan.
Die Neue Klappbrücke der Strauss Bascule Bridge Co.* Müllenhoff. (69) Jan.
Die Auswechselung von Brückentragwerken ohne Verwendung von Gerüsten. Robert Schönhöfer. (52) Jan. 15; (51) Feb. 5.
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Electrical.

Some Economical Applications of the Storage Battery.* J. Lester Woodbridge. (4) Jan.
The Edison Storage Battery.* Harold H. Smith. (4) Jan.
Some Mechanical Points in Armature Construction.* R. Livingstone. (73) Serial beginning Jan. 24.
The Deflection of Armature Shafts.* H. Edington. (11) Jan. 31.
Direct-Current Geared Turbo-Generator Set at Cleveland, Ohio.* (11) Jan. 31.
Municipal Lighting Plant Cost. (60) Feb.
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Performance Similarities of the Direct-Current Shunt Motor and the Polyphase Induction Motor.* William G. Merowitz. (27) Feb. 1.
A New Method of Timing Watt-Hour Meters.* Gordon Thompson. (27) Feb. 1.
Electric Power in Kern River and Midway Oil Fields.* Warren Aikens. (82) Serial beginning Feb. 1.
A Novel German Air Filter (for Electric Generators).* Otto F. Trautmann. (13) Feb. 6.
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A Determination of the International Ampere in Absolute Measure. E. B. Rosa N. E. Dorsey and J. M. Miller. (Abstract of paper from *Bulletin*, U. S. Bureau of Standards.) (73) Feb. 7
Gas-Engine-Driven Central Station, Titusville Light & Power Company. (27 Feb. 8.
International Scandinavian Transmission System. (Comparative costs of generating equipment and proposed 200-mile line from Trollhattau to Copenhagen.) Carl A. Rossander and Georg Centerwall. (Abstract of Report to Swedish Government.) (27) Feb. 8.
A Rational Method of Showing Light Distribution.* Robert French Pierce. (27) Feb. 8.
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Waterbury and Bull's Bridge Plants.* Warren O. Rogers. (64) Feb. 11.
Types of Motors for Steel Mill Auxiliaries. B. R. Shover and E. J. Cheney. (Abstract of paper read before the Am. Assoc. of Iron and Steel Elec. Engrs.) (47) Feb. 14.
An Investigation of Explosion-Proof Motors.* H. H. Clark. (Abstract of paper issued by the U. S. Bureau of Mines.) (73) Feb. 14.

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Electrical—(Continued.)

Inductance and Capacity of Linear Conductors and the Determination of the Capacity of Horizontal Antennæ. Louis Cohen. (73) Serial beginning Feb. 14.
Electric Express Handling at the North Station, Boston.* (27) Feb. 15.
The Central Control of Apparatus in Electric Systems.* (11) Feb. 21.
The Cascade Induction Motor.* M. I. Williams-Ellis. (Abstract of paper read before the South African Inst. of Engrs.) (73) Feb. 21.
Some Methods of Magnifying Feeble Signalling Currents.* S. G. Brown. (Paper read before the Physical Soc.) (73) Feb. 21.
Electric Lighting of Railway Cars. A. L. Broe. (From *General Electric Review.*) (73) Feb. 21.
The Power Plant at Trollhättan, Sweden.* (73) Serial beginning Feb. 21.
Methods of Obtaining Greater Flexibility of Operation in Standard Transformers.* George A. Thornton and George Goldman. (27) Feb. 22.
Electrical Equipment of a Modern Building, Aeolian Hall, New York.* (27) Feb. 22.
Direct-Current Turbo-Generators with Reduction Gears for Railway Service.* (17) Feb. 22.
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Air as an Insulator When in the Presence of Insulating Bodies of Higher Specific Inductive Capacity.* C. L. Fortescue and S. W. Farnsworth. (42) Mar.
The Application of a Theorem of Electrostatics to Insulation Problems. C. Fortescue. (42) Mar.
Thermocouples and Resistance Coils for the Determination of Local Temperatures in Electrical Machines. J. A. Capp and L. T. Robinson. (42) Mar.
Generator and Prime Mover Capacities.* David B. Rushmore and Eric A. Lof. (42) Mar.
The Application of Synchronous Motors to a Water Power Transmission System for the Betterment of Service Standards.* Lucius B. Andrus. (42) Mar.
Electricity in the Packing Industry.* W. D. Bearce. (27) Mar. 1.
Economies in Motor Maintenance. J. C. Thirwall. (Paper read before the Central Elec. Ry. Assoc.) (17) Mar. 1.
A Modern Power Network, the Waterbury-New Britain System.* Charles Rufus Harte. (17) Mar. 1; (27) Mar. 1.
Réseaux de Cables Electriques Souterrains à 30 000 et 60 000 Volts.* (33) Feb. 15.
Die Anlagen der Victoria Falls and Transvaal Power Co. in Südafrika.* G. Klingenberg. (48) Serial beginning Jan. 4.
Ueber Zweck und Anwendung der künstlichen Isolierstoffe bei dem Bau elektrischer Apparate.* H. Passavant. (41) Jan. 23.
Elektrische und Hydraulische Anlagen zur Ausbeutung der Zinnminen in Tekkah.* Herm. Thieme. (41) Serial beginning Jan. 23.
Ueber die Betriebskapazität von Drehstrom-Doppelleitungen.* C. Feldmann und A. C. Loos. (41) Jan. 23.
Störungen von Fernsprechleitungen durch sterngeschaltete Drehstromanlagen ohne und mit Erdung des Generatornullpunktes.* O. Brauns. (41) Serial beginning Jan. 30.
Ueber Verwendung von Eisen in elektrischen Messinstrumenten.* M. Dolivo-Dobrowolsky. (41) Jan. 30.
Ueberspannungen und Ueberspannungsschutz. W. Petersen. (41) Serial beginning Feb. 13.
Die Anwendung der Elektrizität in Brauereien.* G. Bachmann. (41) Serial beginning Feb. 13.
Ueber die Entstehung und Unterdrückung selbsterregter Ströme in Drehstrom-Reihenschlussmaschinen.* L. Binder und E. Dyhr. (41) Serial beginning Feb. 20.
Nachteile einphasiger Verlegung von Wechselstromleitungen in Rohren. L. Bloch. (41) Feb. 20.
Einiges über Vorzeichen Richtungsbestimmung und Wicklungssinne.* A. Scherbius. (41) Feb. 20.

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The Brazilian Battleship *Rio de Janeiro*.* (11) Jan. 24.
The Curtis Turbines of the Scouts *Marsala* and *Nino Bixio*. Vittorio Malfatti. (Translation in abstract of paper read before the Collegio degli Ingegneri Navali e Meccanici in Italia.) (11) Jan. 31.
The Diesel Engine as Motive Power in the Merchant Marine, with Special Reference to the First Successful Motor Ship, *Christian X*. Ole K. Olsen. (Paper read before the Louisiana Eng. Soc.) (1) Feb.
Great Lakes Steamers for Coastwise Service.* (95) Feb.
The Possibilities of the Marine Oil Engine.* Theodore Lucas. (95) Feb.
The American-Built Steam Yacht *Lydonia*.* A. S. Reed, Jr. (95) Feb.
Marine Turbine Operation and Economy. H. C. Dinger. (95) Serial beginning Feb.

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The Uses of Gases for Fire Extinction.* (On ships.) E. Kilburn Scott. (Abstract from paper read before the Inst. of Marine Engrs.) (57) Feb. 7.
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The Unsinkability of Modern Sea-Going Ships.* Flamm. (11) Feb. 14.
The Unsinkable Ship (Wotherspoon's System).* (12) Feb. 14.
Shipbuilding Berth Gantries at the Nagasaki Yard.* (11) Feb. 14.
Two-Stroke Cycle Continuous-Combustion Oil Engines for Ships.* (47) Feb. 21.
The Heavy-Oil Engine in Marine Service.* R. Clifford H. Reid. (19) Feb. 22.
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Der neue Längshelling auf der Königlichen Werft in Emden.* Paulmann und Blaum. (49) Pt. 1.
Der Unfall und die Wiederherstellung von Dock V auf der Kaiserlichen Werft in Kiel.* Behrendt und O. Franzius. (49) Pt. 10, 1912.
Das Dieselschiff *Rolandseck*.* W. Kaemmerer. (48) Jan. 4.
Schiffshebewerk mit Seitenschwimmern und stellbaren Riegeln.* Fr. Jebens. (51) Feb. 19.

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Rolling-Mill Practice in the United States.* J. Puppe. (71) Vol. 186.
A New Type and Method of Construction of Large Gas-Engines.* Alan E. L. Chorlton. (71) Vol. 186.
Aerial Flight.* Henry Reginald Arnulph Mallock. (63) Vol. 190.
Theory of Steam Accumulators and Regenerative Processes.* F. G. Gasche. (58) Dec.
Methods and Means of Smoke Abatement. O. R. McBride. (2) Jan.
Mechanical Haulage. John Robinson. (Paper read before the Inst. of Mun. Engrs.) (104) Jan. 24.
Air Friction.* A. P. Thurston. (11) Jan. 24.
Briquetting Plant for Metal Turnings and Borings.* (11) Jan. 24.
The Determination of Water in Coal. G. N. Huntly and J. H. Coste. (Abstract of paper read before the Soc. of Chemical Industry.) (57) Jan. 24.
Two Large German Installations of Koppers Coke Ovens.* E. M. Hann. (Paper read before the South Wales Inst. of Engrs.) (57) Jan. 24.
Surface Combustion. William Arthur Bone. (Paper read before the German Chemical Soc.) (66) Jan. 28.
The Humphrey Pump.* H. de P. Birkett. (Paper read before the London and Southern District Junior Gas Assoc.) (66) Jan. 28.
Development of the Hydroplane.* Alessandro Guidoni. (Translation in abstract of paper read before the Collegio degli Ingegneri Navali e Meccanici, in Italia.) (11) Jan. 31.
The Oddie "Simplex" Feed-Pumps and Air-Pumps.* (11) Jan. 31.
The Williams-Janney Variable-Speed Gear.* (11) Jan. 31.
Stability Devices for Aeroplanes.* Mervyn O'Gorman. (Abstract of paper read before the Aeronautical Soc. of Great Britain.) (11) Jan. 31.
The Design and Construction of Shop Cranes. Frank W. Suffield. (Abstract of paper read before the Birmingham Assoc. of Mech. Engrs.) (47) Jan. 31.
The Constitution of Portland Cement. (Paper read before the National Assoc. of Cement Users.) P. H. Bates. (67) Feb.
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Making Small Parts on Screw Machines.* (20) Feb. 6.
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The Gyroscopic Action of Rotary Motors in Aeroplanes. Herbert Chatley. (11) Feb. 7.
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Supplying and Vaporizing Fuel for Oil Engines.* (47) Feb. 7.
High-Speed Bearings.* John C. K. Balfry. (Paper read before the Rugby Eng. Soc.) (47) Feb. 7.
Illumination of Engineering Workshops by Gas, Electricity, and Oil.* Franklin Thorp. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Feb. 7; (66) Feb. 4.
The Determination of Water in Coal. R. Lessing. (Paper read before the Inter. Congress of Applied Chemistry.) (57) Serial beginning Feb. 7.
The Autostable, A Flying Machine Possessing a High Degree of Automatic Stability.* W. I. Chambers. (19) Feb. 8.
New Coal Handling Plant at Fort William, Ont., Canadian Pacific Ry.* (18) Feb. 8.
Manufacturing the Circular Cut File.* Chester L. Lucas. (19) Feb. 8.
Weighing Ammonia to Determine Refrigerating Capacity.* Fred Ophüls. (64) Feb. 11.
Coke Quenching for Chamber Ovens.* Wilhelm Reubold. (Translation from *Journal für Gasbeleuchtung.*) (66) Feb. 11.
Gas for Heat, Power, and Light. T. Carmichael. (Paper read before the Scottish Junior Gas Assoc.) (66) Feb. 11.
Every-Day Experiences in Gas Distribution. John Ferguson. (Paper read before the Scottish Junior Gas Assoc.) (66) Feb. 11.
The Balance Draft System of Furnace Regulation.* (64) Feb. 11.
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A Study of Motor Truck Operation and Economy in City Sale and Delivery of Sand and Crushed Stone. R. W. Hutchinson, Jr. (86) Feb. 12.
A Jack Screw for Pulling Stuck Drills and Sounding Rods.* Charles C. Mitchell. (86) Feb. 12.
Boiler Explosions as Affected by Unsymmetrical Riveted Joints.* S. H. Barraclough and others. (Abstract of *Proceedings*, Eng. Assoc. of New South Wales.) (13) Feb. 13.
The Laws of Similitude (Relates to Aeronautics).* L. Bairstow. (11) Feb. 14.
New Sources of Supply for the Manufacture of Paper.* Clayton Beadle and Henry P. Stevens. (29) Feb. 14.
Power from Kerosene.* L. W. Ellis and W. R. Dray. (19) Feb. 15.
Isolated Plant of Gasoline Engine Factory.* Harry C. Spillman. (27) Feb. 15.
Oil-Piping Layout for Transformers Using Thin Oil.* Fred Bush. (27) Feb. 15.
The General Gas-Flow Formulæ Computer.* L. Meissner. (83) Feb. 15.
Effect of Diluents on the Candle-Power of Oil-Gas. Arthur H. Elliott. (Paper read before the Am. Gas. Inst.) (83) Feb. 15.
The Field for Small Steam Turbines. W. J. A. Loudon and Ashley P. Peck. (Abstract of paper read before the Assoc. of Ry. Elec. Engrs.) (64) Feb. 18.
The Carbonization of Gas Coal in Large Quantities. (Translation from *Gazzetta Chimica Italiana.*) (66) Feb. 18.
Amount of Heat Obtainable from Coal Residuals.* Henry O'Connor. (66) Feb. 18.
American Investigation on Calorimetry.* (Report of Committee of the Am. Gas. Inst.) (66) Feb. 18.
Gas Manufacture, Gas-Works Construction, and Management. J. G. Newbigging. (Paper read before the Manchester Univ.) (66) Feb. 18.
Costs of Producing Power Based on Iowa Coals, with a Comparison of Estimated Costs and Costs from Actual Tests.* (86) Feb. 19.
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Coaling Plants at Panama Canal. (From the *Canal Record.*) (62) Feb. 24.
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Some Notes on Charging and Discharging Machines for Retorts. W. L. Heald. (Paper read before the Manchester and District Junior Gas Assoc.) (24) Feb. 24; (66) Feb. 4.
Large High Vacuum Condenser Test.* J. J. Brown. (64) Feb. 25.
Considerations When Applying Motor Drives.* A. G. Popcke. (72) Feb. 27.
Special Machine for Winding Coils.* (72) Feb. 27.
Cost of Ice Making in Small Plants. R. P. Kehoe. (From *Pure Products.*) (19) Mar. 1.
Arcadia Coal Company's Steam Turbine Plant.* (27) Mar. 1.
Combination Steam-Electric Ice-Making Plant in Chicago.* (27) Mar. 1.
Report of Committee on Consumers' Meters of American Gas Inst. Charles C. Schiller. (24) Mar. 3.
Oil as a Fuel.* Reginald Trautschold. (64) Mar. 4.
Turbines and Diesel Engines in Central Stations.* M. Gercke. (Paper read before the Assoc. of Elec. Central-Station Managers and Engrs., Kiel, Germany.) (64) Mar. 4.
Smoke Prevention. A. C. Wilson. (64) Mar. 4.
Power Transmission by Compressed Air. George Barr. (Paper read before the Manchester Assoc. of Engrs.) (64) Mar. 4.
Production Simultanée d'Energie Electrique et de Chaleur, leur Distination pour des Buts Industriels ou Urbains.* A. Beaurrienne. (32) Nov.
Les Gaz de Fours à Coke, leur Utilisation, leurs Applications.* A. Gouvy. (32) Dec.
IIe Congrès National du Froid à Toulouse, en Septembre 1912; Compte Rendu. E. Barbet. (32) Dec.
La Résistance de l'Air et les Expériences sur les Modèles Réduits.* E. Jouguet. (37) Jan. 31.
Pelles à Vapeur.* (34) Feb.
Presses à Forger, Système Haniel et Lueg.* (34) Feb.
Four à Tunnel, Système Dressler, pour la Cuisson des Produits Céramiques.* (35) Feb.
Pompe à Incendie Automobile Multicylindrique Système Drouville.* D. Duaner. (33) Feb. 1.
Les Procédés Modernes d'Agglomération des Minerais de Fer. Henry Martin. (33) Serial beginning Feb. 15.
Trockenbagger mit Seitenförderer.* Ostmann. (49) Pt. 10, 1912.
Neuere Bestrebungen im Dampfkesselbau.* Friedrich Münzinger. (48) Dec. 21.
Zur Theorie der Riementriebe.* W. Maier. (48) Dec. 21.
Verbrauchsversuch an einem Luftdruckhammer mit nur einem Zylinder.* Otto Fuchs. (48) Dec. 28.
Die Subventions-Motorlastwagen der österreichischen Heeresverwaltung.* A. Heller. (48) Dec. 28.
Die Anlagen der Victoria Falls and Transvaal Power Co. in Südafrika.* G. Klingenberg. (48) Serial beginning Jan. 4.
Ueber das Absaugen der Rohgase bei dem Koksofenbetrieb.* E. Jenkner. (50) Jan. 23.
Das Bessemerwerk der Königshütte (ein Gedenkblatt zum 100-jährigen Geburtstage Henry Bessemers).* Hermann Jllies. (50) Feb. 6.
Ueber die Zersetzungstemperatur von Koksofengas.* Oskar Simmersbach. (50) Feb. 6.
Der elektrische Antrieb von Blechscheren, Richt und Biegemaschinen.* (50) Feb. 13.

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Method of Producing Sound Ingots.* Robert Hadfield (71) Vol. 186.
A New Method for the Improvement of the Soundness of Steel Ingots by the Aid of Thermit.* Hans Goldschmidt. (71) Vol. 186.
On Allotropy in General and That of Iron in Particular.* Carl Benedicks. (71) Vol. 186.
The Magnetic Properties of Manganese and Nickel Steels.* S. Hilpert and W. Mathesius. (71) Vol. 186.
Does Commercial Hyper-Eutectic White Iron Free From Manganese Exist? Henry M. Howe. (71) Vol. 186.
The Thermal-Magnetic Transformations of 25 per cent. Nickel Steel.* E. Colver-Glanert and S. Hilpert. (71) Vol. 186.
Iron and Nitrogen.* J. H. Andrew. (71) Vol. 186.

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The Gases Evolved on Heating Steel to Its Melting Point in a Vacuum.* G. Wesley Austin. (71) Vol. 186.
Manufacture of Open-Hearth Steel With Reference to Improvement in Yield. F. W. Paul. (71) Vol. 186.
Steelworks Yields. Percy Longmuir and W. H. Robinson. (71) Vol. 186.
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Sudbury Nickel-Copper Industry.* Reginald E. Hore. (45) Feb.
Automatic Sand Sluicer.* (For Cyanide Mills.) (45) Feb.
The Woodbury Slime Classifier.* Edward T. Wright. (45) Feb.
Recent Advances in the Spray Process for the Production of Metallic Coatings.* M. V. Schoop. (105) Feb.
Resistivity of a Few Metals Through a Wide Range of Temperature. Edwin F. Northrup and V. A. Suydam. (3) Feb.
The First Combination Induction Furnace in Operation in the United States.* C. H. von Baur. (105) Feb.
An Analysis of Steel Ingot Manufacture, The Influence of Common Defects on the Quality of Rails and Structural Material and Means for Correcting Them.* Bradley Stoughton. (15) Feb. 7.
American Ingot Iron vs. Steel. G. F. Ahlbrandt. (Abstract from paper read before the Cleveland Eng. Soc.) (62) Feb. 10.
A Modern Design of Blastfurnace.* (47) Feb. 14.
Elevating Pulp.* M. W. von Bernewitz. (103) Feb. 15.
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The Krupp Steel Works Laboratories.* Alfred Gradenwitz. (20) Feb. 20.
The Reactions of the Puddling Process. Thomas Turner. (Paper read before the West of Scotland Iron and Steel Inst.) (47) Feb. 21; (11) Feb. 14.
Developments in the Preparation of Iron Ores. J. W. H. Hamilton. (Paper read before the Am. Iron and Steel Inst.) (47) Serial beginning Feb. 21.
The Blair Indestructible Port and Bulkhead for Open-Hearth Furnaces.* (22) Feb. 21.
Silver Cyanidation at Tonopah.* Herbert A. Megraw. (16) Serial beginning Feb. 22.
Regrinding at the Pittsburg Silver Peak Mill.* S. J. Kidder. (103) Feb. 22.
A Volatilisation Process for Gold Extraction. Ben Howe. (From *Journal,* Western Australian Chamber of Mines.) (68) Feb. 22.
Steel Castings Without the Use of Manganese.* Edwin F. Cone. (20) Feb. 27.
Les Hauts-Fourneaux de Caen.* Robert le Chatelier. (93) Feb.
Disposition Générale de l'Usine de Caen.* de Loisy. (93) Feb.
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Experimentelle Untersuchung des Kupolofen-Schmelzprozesses.* Friedrich Hüser. (50) Jan. 30.
Grosse elektrische Oefen, Bauart Helfenstein.* Max Oesterreich. (50) Feb. 20.
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Gob-Fires and the Prevention of Gob-Fires in Mines. John Harger. (Paper read before the Manchester Geological and Min. Soc.) (106) Vol. 44, Pt. 2.
The South-Eastern Coal-Field, the Associated Rocks, and the Buried Plateau. W. Boyd Dawkins. (106) Vol. 44, Pt. 2.
Notes on Haulage-Clips in Use in North Staffordshire. W. G. Salt and A. L. Lovatt. (Paper read before the North Staffordshire Inst. of Min. and Mech. Engrs.) (106) Vol. 44, Pt. 2.
Notes on Gob-Fires Near Ashby-De-La-Zouch, Leicestershire. Thomas Stubbs. (Paper read before the Midland Inst. of Min., Civ., and Mech. Engrs.) (106) Vol. 44, Pt. 2.
Mine-Rescue Appliances: A Danger Occurring in the Use of Apparatus in Which an Injector is Employed. John Cadman. (Paper read before the South Staffordshire and Warwickshire Inst. of Min. Engrs.) (106) Vol. 44, Pt. 2.
The Iron Ore Resources of Chili with a Note on the Corral Iron and Steel Works.* Charles Vattier. (71) Vol. 186.
Water Haigh Colliery, Woodlesford.* (22) Jan. 24.
Chemical Interpolation of Anthracite. M. S. Hachita. (Paper read before the Lackawanna Chemical Soc.) (45) Feb.
Timber-Framing Mills for Square-Set Mines.* Claude T. Rice. (82) Serial beginning Feb. 1.

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Electrically Equipped Coal Mines in Nova Scotia.* C. H. Wright. (27) Feb. 1.
The Thury System of Electrical Transmission of Power for Mines.* Sydney F. Walker. (22) Feb. 7.
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Electric Hoists for Mine Service.* H. A. Russell. (Paper read before the California Miners Assoc.) (103) Feb. 8.
The Taranaki Petroleum Field. E. de Courcy Clarke. (Abstract from Report of New Zealand Geological Soc.) (68) Serial beginning Feb. 8.
Sinking a Circular Shaft.* E. Mackay Heriot. (16) Serial beginning Feb. 8.
Development of the Michipicoten Range, Ontario.* George E. Edwards. (82) Feb. 15.
Ventilation Standard and Test at Metal Mines. Frank Reed. (Abstract of paper read before the Australasian Inst. of Min. Engrs.) (82) Feb. 15.
Adoption of Standard Screens for Screen Analysis. Robert H. Richards. (82) Feb. 15.
Explosion-Proof Electric Equipment for Mining Service.* (13) Feb. 20.
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The Development of Mechanical Power in the Mines of the North Eastern Coalfield: A Comparison and a Contrast. Robert Nelson. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Feb. 21.
A New Gasoline Rock Drill.* (18) Feb. 22; (57) Feb. 21.
The Mother Lode Region, California.* Lewis H. Eddy. (16) Feb. 22.
A Duplex Rock Channeler with Turntable. (13) Feb. 27.
Electric Power on the Michigan Copper Range.* George E. Edwards. (82) Mar. 1.

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Chemical Production of Light. Wilder D. Bancroft. (3) Feb.
The Economic and Hygienic Value of Good Illumination.* Leon Gaster. (29) Feb. 7.
The Measurement of Light and Illumination. J. S. Dow. (Paper read at the Polytechnic, London.) (24) Serial beginning Feb. 17.
Cost-Estimating and Rate-Fixing in the General Shop. (11) Serial beginning Feb. 7.
Atmospheric Humidity.* John Thompson. (Paper read before the National Assoc. of Colliery Managers.) (22) Feb. 14.
Photographing Sound.* Arthur L. Foley and Wilmer H. Souder. (19) Feb. 15.
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A New Entropy Analog.* V. Karapetoff. (From *General Electrical Review.*) (13) Feb. 27.
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Park Drives and Boulevards.* Linn White. (60) Feb.
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The Petrographic Study of Road Building Rocks in the U. S. Office of Public Roads.* (86) Feb. 5.
Municipal Plant for the Storage of Road Oil.* (86) Feb. 5.
The Consistency of Bituminous Materials—Its Determination and Value in Specifications. W. W. Crosby, M. Am. Soc. C. E. (Paper read before Am. Assoc. for the Advancement of Science.) (86) Feb. 5.
How Nearly Does the Modern Yellow Pine Block Pavement Approach to the Ideal Pavement and What Improvements Can We Suggest? H. L. Collier. (Abstract of paper read before the Am. Wood Preservers' Assoc.) (96) Feb. 6.
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Modern Road-Making Machinery and its Use. T. R. Agg. (Abstract of paper read before the Ill. Soc. of Engrs. and Surveyors.) (13) Feb. 13; (86) Feb. 5.
Tests of Road Metal. C. W. L. Alexander, Assoc. M. Inst. C. E. (104) Feb. 14.

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Bituminous Surfaces on Brick Pavements. Ellis R. Dutton. (Abstract of paper read before the Am. Assoc. for the Advancement of Sci.) (14) Feb. 15; (86) Feb. 19.
Some Observations and Experiments on the Tractive Power of Horses. (86) Feb. 19.
Limitations in the Use of Bituminous Carpet Surfaces.* Arthur W. Dean, M. Am. Soc. C. E. (Paper read before Am. Assoc. for the Advancement of Sci.) (86) Feb. 19.
Cost of Street Cleaning, Sprinkling and Oiling at Boston, Mass., in 1911. (86) Feb. 19.
Method and Cost of Clearing Site, Grading Streets and Constructing Concrete Sidewalks for Real Estate Development on Long Island. Myron H. Lewis. (86) Feb. 19.
Service Test of Street Pavements under City Traffic. (86) Feb. 19.
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London Street Pavings, Metropolitan Committee's Tenth Annual Report. (104) Feb. 21.
A Mile of Test Pavements on Second Avenue, New York. H. W. Durham. (14) Feb. 22.
Relative Advantages of Laying Brick Pavements on Sand Foundations and Cement Concrete Foundations. Robert Hoffman, M. Am. Soc. C. E. (Abstract of paper read before Am. Assoc. for the Advancement of Sci.) (86) Feb. 26.
Some Conditions Affecting the Interaction of Motor Vehicle Wheels and Road Surfaces. L. I. Hewes. (Paper read before Am. Assoc. for the Advancement of Sci.) (86) Feb. 26.
Tar Macadam Construction by the Board of Public Roads of Rhode Island in 1912.* Herbert C. Peore. (86) Feb. 26.
The City Traffic Census as Conducted by the Bureau of Highways, Borough of Brooklyn, New York City. W. H. Messenger. (Abstract of paper read before Am. Assoc. for the Advancement of Sci.) (86) Feb. 26.
Brick Pavements for Country Roads.* William C. Perkins. (Paper read before Am. Assoc. for the Advancement of Sci.) (96) Feb. 27; (60) Feb.; (86) Feb. 19.
Concrete Paving Between Car Tracks in Birmingham.* (14) Mar. 1.
Cost Data on Street Paving. W. W. Kerch. (Abstract of paper read before the Ill. Soc. of Engrs. and Surveyors.) (14) Mar. 1.
Note sur les Travaux Entrepris pour Protéger la Route National No. 12 d'Alger à la Frontière Tunisienne contre l'Envabissement des Sables.* M. Daujon. (43) Nov.
Die Versorgung von Städten mit Koksofengas und die kommunalpolitische Seite dieser Frage. R. Nübling. (39) Feb. 5.
Tafeln für Zinseszins und Rentenrechnung zur Berechnung des Gestehungspreises zu unterhaltender Asphaltstrassen. Brehmer. (39) Feb. 5.

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The Use of Single-Phase Commutator Motors for Electric Traction on Long-Distance Railways.* Stanley Parker Smith. (63) Vol. 190.
The London, Brighton and South Coast Railway Electrification.* (73) Jan. 31.
Electrification on the Midi Railway.* (12) Serial beginning Jan. 31.
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Curves of Locomotive Operation.* L. R. Pomeroy. (25) Feb.
The Bulldozer in Railway Shops.* Lewis D. Freeman. (25) Feb.
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Adaptability of Concrete to Track Elevation Work. Alfred W. Hoffman. (87) Feb.
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Heat-Treated Chrome Vanadium Tires.* (108) Feb.
Adhesion and Rack Locomotives on the Italian State Railway.* Luigi Velani. (From *Revista tecnica delle ferrovie Italiane.*) (88) Feb.
Historical Sketch of the Beach Hydraulic Shield.* Frederick C. Beach. (36) Feb.
The Electro-Pneumatic Brake. N. A. Campbell. (65) Feb.
The Manufacture of Chilled Iron Car Wheels. Frederick C. Weber. (105) Feb.
Pacific Type Locomotives for the Seaboard Air Line.* (18) Feb. 1.
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Some Features of Grade Separation by Joint Track Elevation in Joliet, Illinois. R. A. Crook. (Abstract of paper read before Illinois Soc. of Engrs. and Surveyors.) (86) Feb. 5.
The New Gas-Electric Locomotive of the Minneapolis, St. Paul, Rochester & Dubuque Electric Traction Company.* (86) Feb. 5.
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The Uintah Railway Line with Steep Grades and Sharp Curves.* E. A. Porter. (14) Feb. 15.
Factors Affecting Derailments on Sharp Curves. (14) Feb. 15.
An Electric Tractor for Hauling Freight Cars in City Streets; Pennsylvania R. R.* (13) Feb. 20.
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The Mittenwald Single-Phase Railway.* (12) Serial beginning Feb. 21.
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Die Bahnlinie Ebnat-Nesslau.* A. Acatos. (107) Serial beginning Feb. 1.
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Ueber die Kraftwerksausnützung beim Zukünftigen elektrischen Betrieb der Schweizerischen Eisenbahn.* W. Kummer. (107) Feb. 15.

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Discussion on Rates and Fares. William J. Clark. (Paper read before the Am. Elec. Ry. Assoc.) (17) Feb. 1.
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Recommendations for San Francisco Cars. B. J. Arnold. (Abstract of Supplement to report on the design of the new United Railroads' motor car.) (17) Feb. 8.
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Report on Cincinnati Terminal Possibilities.* Bion J. Arnold. (Report to the Cincinnati Rapid Transit Comm.) (17) Feb. 22.
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Municipal Plants for Electrolytic Sewage Purification.* C. L. Edholm. (60) Feb.
Use of Pitot Tube in Air Measurements.* Frank L. Busey. (64) Feb. 4.
The Human Element in Industry, Economics of Proper Attention to Shop Hygiene Through a Service Department Approved Methods of Ventilation. Winthrop Talbot. (20) Serial beginning Feb. 6.
Drop-Feed Hot-Water Heating System.* (101) Feb. 7.
The Possibilities of House Heating by Artificial Gas as a Fuel. A. F. Krippner. (Paper read before the National Commercial Gas Assoc.) (24) Feb. 10; (66) Feb. 11.
Methods of Garbage Disposal Applicable to Conditions in the Smaller Cities, with Some Costs and Other Data. Samuel A. Greeley. (Paper read before Illinois Soc. of Engrs. and Surveyors.) (86) Feb. 12; (14) Feb. 15.
Salient Features of Modern Refuse Destructor Practice in Great Britain. James A. Seager. (86) Feb. 12.
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Sewage Disposal Investigations at Cleveland.* R. Winthrop Pratt. (13) Feb. 13.
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Facts and Fancies About Sewage Disposal. Gilbert Thomson. (Abstract of paper read before the Sanitary Assoc. of Scotland.) (96) Feb. 20.
A Plea for Scientific Research in the Field of Sewage Purification. A. J. Martin. (Paper read before the Inst. of San. Engrs.) (104) Feb. 21.
Combination Heating System in a Residence.* Frank K. Chew. (Paper read before the Am. Soc. of Heating and Ventilating Engrs.) (101) Feb. 21.
A Concrete Wash-Stand.* Frank A. Bowman. (103) Feb. 22.
Sewage-Pumping Plant in Kansas City.* O. L. Eltinge. (14) Feb. 22.
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Sludge Disposal. Karl Imhoff. (Paper read before the Inter. Congress on Hygiene and Demography.) (96) Feb. 27.
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Street Cleaning Costs at Spokane, Wash., in 1912. S. A. Levington. (13) Feb. 27.
Specifications for Drain Tile and Sewer Pipe. (Report of Comm., Iowa State Drainage Assoc.) (14) Mar. 1.
Sewage-Disposal Plant at the Great Lakes Naval Training Station, Treating Sewage by Reduction Tanks and Anaërobic and Sprinkling Filters.* (14) Mar. 1.

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Structure of Galvanized Iron.* W. Arthur and W. H. Walker. (Abstract of paper read before the Am. Inst. of Metals.) (22) Jan. 24.
Breakdown Tests of Metals.* O. Boudouard. (Paper read before the Inter. Assoc. for Testing Materials.) (47) Jan. 24.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XXXIX. MARCH, 1913. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

KINETIC EFFECTS OF CROWDS.

By C. J. Tilden, Assoc. M. Am. Soc. C. E.
To be Presented April 16th, 1913.

For nearly a century the accepted value for the weight of a dense crowd of people has been about 100 lb. per sq. ft., conservative designers often assuming a slightly higher figure, and their more daring brethren a considerably lower one.* The investigations of Stoney,† Kernot,‡ and others, long ago showed that this was by no means the maximum value, and the elaborate work of Johnson,§ published in 1904, showed that an intensity of 183 lb. per sq. ft. was within the range of possibility. The effect of Johnson's investigations was to raise slightly the load intensities prescribed by some specifications, but this effect was by no means general.

Thus far, the purely static effect of a crowd is the only one that has received careful study by engineers, and the "dead weight" is the loading assumed. The fact that this is not sufficient in considering the load that may come on a bridge or other structure was

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* "A Memoir on Suspension Bridges." Charles S. Drewry, London, 1832, p. 113.
"Work on Bridge Building." S. Whipple, Utica, N. Y., 1847, Essay No. II, p. 115.
"Encyclopedia of Architecture." Gwilt, p. 429.
For modern load requirements in specifications of highway bridges, etc., see *Transactions*, Am. Soc. C. E., Vol. LIV, p. 479; *Engineering News*, Vol. 65, p. 392, pp. 440 and 460; Vol. 66, pp. 61 and 643; Vol. 68, p. 824, etc., etc.; Illinois Highway Commission, Third Report, p. 121.

† "Theory of Strains in Girders and Similar Structures." B. B. Stoney, London, Revised Edition, 1886, p. 616.

‡ See Editorial, *Engineering News*, Vol. 29 (March 16th, 1893), p. 252.

§ *Engineering News*, Vol. 51, p. 360; *Transactions*, Am. Soc. C. E., Vol. LIV, p. 441; *Harvard Engineering Journal*, Vol. IV (April, 1905), p. 29.

recognized by Robert Stevenson in the early part of the last century. In his paper entitled "An Account of Suspension Bridges", published in 1821,* is the following interesting paragraph:

"But the effect we have to provide against in bridges of suspension, is not merely what is technically termed dead weight. A more powerful agent exists in the sudden impulses, or jerking motion of the load, * * * The greatest trial, for example, which the timber bridge at Montrose, about 500 ft. in extent, has been considered to withstand, is the passing of a regiment of foot, marching in regular time. A troop of cavalry, on the contrary, does not produce corresponding effects, owing to the irregular step of the horses. The same observations apply to a crowd of persons walking promiscuously, or a drove of cattle," etc.

Stevenson, however, does not suggest any definite values for this "more powerful agent" which he recognizes and describes with such clear appreciation.

Shortly after Johnson's paper was published, a letter signed by Mr. R. Moreland appeared in London *Engineering*,† in which a suggestive experiment was reported:

"In 1900 my firm had a contract for the Manchester Racecourse for some large stands, and to gain information we put as many men as possible on our 10-ton weigh-bridge, and we found that we could get ninety men as close as possible into a space 14 ft. by 8 ft., or 112 square feet, and they weighed 115 cwt. 1 qr., which would equal 1 cwt. per square foot. We then asked them to jump, and the load went up to 1½ cwt.; they then ran four abreast across the machine, but no excess was recorded over the 1½ cwt. per square foot."

Details of this experiment are meager, but it is the only one, so far as the writer is aware, in which a determination is sought of the increased load effect due to motion in a group of human beings. In general, in structural work at least, this increase is assumed to be cared for by the "factor of safety", that pernicious and misnamed offspring of ignorance.

The study of kinetic effect is far more complex than that of static effect, for the movements of a crowd of people may take place in infinite variety, and each change of motion must be accompanied by some exertion of force other than the purely static condition. The problems suggested, therefore, are capable, at best, of only approximate solution. The following study is an attempt to throw some light on

* *Edinburgh Philosophical Journal*, No. 10, p. 255.

† Vol. LXXIX (April 28th, 1905), p. 551.

the question of determining and analyzing as far as possible the forces exerted by an individual under certain simple conditions of motion. The extent to which such observations apply to crowds, and the limitations of such an application are then considered.

The subject is divided naturally into two parts: first, the vertical effect, as in the experiment described by Moreland and quoted above, manifested as an increase over the static or dead load; and, secondly, the horizontal effect which appears as a lateral force. The first is due to changes of motion in a vertical direction, the second to similar changes in a horizontal direction. The experiments described are simple and direct, and though the results may not be scientifically exact, they are at least instructive, and furnish a basis for modifying somewhat our views on the possible effect of crowds.

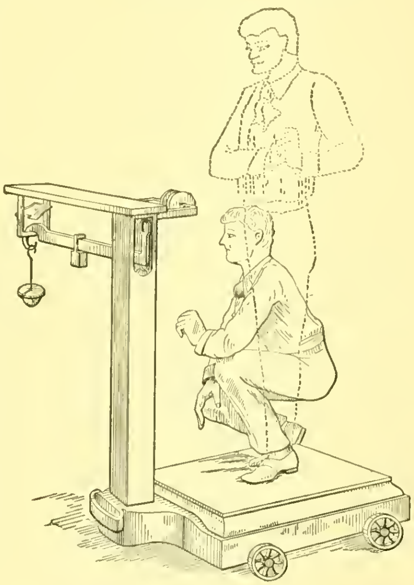

FIG. 1.

Vertical Effect.—1.—Rising Suddenly from a Crouching Posture.—In this experiment the subject was asked to assume a crouching position, as shown by Fig. 1, on an ordinary platform scale. His weight was determined and recorded. The counterpoise was then moved out on the scale-beam until it registered a load 40 or 50 lb. in excess of the static load, the hooked end of the scale-beam resting, of course, against the lower stop. The subject was then asked to rise smartly to a standing position, as shown by the dotted lines in Fig. 1, whereupon the scale-beam shot up momentarily, showing a sudden large increase in the downward load effect of the man, as a result of the upward acceleration given to his body. A somewhat larger load was then registered on the scale-beam and the operation repeated. A great deal of care was exercised to prevent the motion from becoming a jump, and the subject was warned not to let his feet leave the platform. The movement was smart and quick, however, occupying, per-

haps, $\frac{1}{3}$ or $\frac{1}{4}$ sec. Each repetition of the movement was recorded, and the words "Raised" or "Not raised", referring to the scale-beam, were written after the number representing the load registered on the scales. The following is a sample of the record sheet kept of each experiment:

(Name of subject.) Static weight = 155 lb. (Date.)

Scales set at 220 lb.	Raised.
" " " 240 "	Raised.
" " " 260 "	Barely raised?
" " " 260 "	Raised.
" " " 265 "	Raised.
" " " 270 "	Raised.
" " " 275 "	Not raised.
" " " 275 "	Raised.
" " " 280 "	?
" " " 280 "	Not raised.
" " " 280 "	Barely raised?

It is seen that this man, weighing 155 lb., exerted a momentary downward pressure of 275 lb., as shown by the scale-beam lifting from the stop, as a result of the movement described. As there was some question, even after three trials, as to whether 280 lb. had been exerted, the figure taken was 275 lb. or an increase of 77% over the static load. (See Line D, Fig. 2, I.) Eight experiments of this sort were tried on seven different men, weighing from 139 to 233 lb. The results are shown graphically by Fig. 2, I, in which the black portion of each line represents the static weight and the shaded portion the increase due to rising suddenly from a crouching position. It is interesting to note that the greatest increase recorded, 80%, occurs for the heaviest man, *G*, a former tennis champion who is very active on his feet.

The two lines marked A and A_x, one showing 58% increase and the other 67%, are for the same man. The result shown at A was obtained in the usual way, as illustrated by Fig. 1 and previously described. The case, A_x, differed from A in that a timber, about 11 in. deep and weighing 108 lb., was put on the scale platform on which the subject stood. While it is not likely that the interposition of this heavy mass had anything to do with the slight increase (from 58 to 67%) shown in A_x, it obviously did not tend to diminish the kinetic effect.

II.—Rising Suddenly from a Sitting Position.—This experiment was similar to I, and was conducted in the same manner, the only difference being that the subject rose suddenly from the chair in which he was seated, instead of getting up from a crouching position.

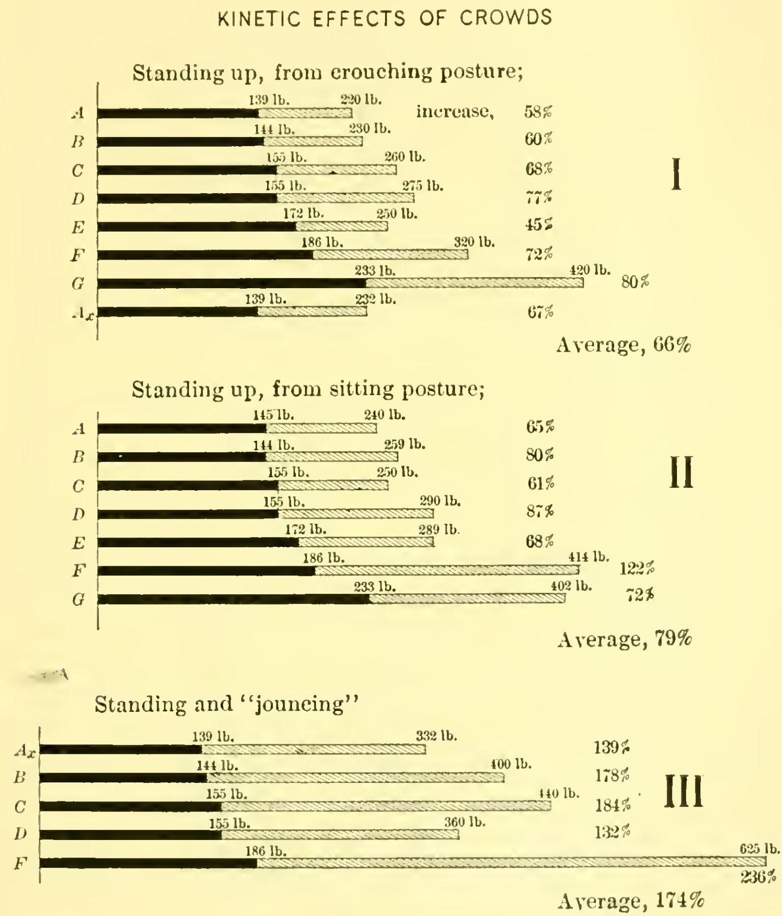

FIG. 2.

The movement is indicated clearly by the photographs, Plate XLI, Figs. 1 and 2, in which the two successive positions of the subject are shown by a double exposure of the negative. The records were made in precisely the same manner as for Experiment I, with the results as shown in Fig. 2, II. The letters, *A, B, C,* etc., refer to the same in-

dividuals as in I. The following is a sample "log-sheet" for this experiment, being the record for *B:*

(Name of subject.) Scale reading = 155 lb. (Date.)
Weight of chair = 11 lb.

Static weight of subject = 144 lb.

Scales set at 200 lb............Raised.
" " " 220 "............Raised.
" " " 240 "............Raised.
" " " 250 "............Raised.
" " " 260 "............Not raised.
" " " 260 "............Not raised.
" " " 260 "............Raised—third trial.
" " " 265 "............Raised—first trial.
" " " 270 "............Raised—first trial.
" " " 275 "............Not raised in three trials.
270 — 11 = 259 lb., total net effect, static + kinetic;
= 115 lb. increase, or 80 per cent.

It is interesting to note that this movement, which may readily occur in a theater, grandstand, or other place where a large audience is gathered, gives distinctly higher values than are found in Experiment I, the average increase being nearly 80%, and no individual case, of the seven recorded, falling below 60 per cent.

III.—Jouncing.—In this experiment, the results of which are shown by III, Fig. 2, an attempt was made to get an approach to a maximum individual effect. The subject, standing on the scale platform, suddenly bent the knees and as quickly straightened them again, at the same time jerking the arms and shoulders downward to intensify the effort exerted. The instructions were to get as vigorous a movement as possible without letting the feet leave the scale platform. The vertical movement of the body did not probably exceed 2 or 3 in. The experiment is interesting and suggestive, but hardly of much importance, as the time during which the greatest effort acts is necessarily exceedingly short. The effort is more in the nature of a quick blow. Its bearing in relation to I and II will be discussed later.

Horizontal Effect.—I.—Some idea of the horizontal effort exerted in Experiment II (rising from a sitting posture) may be obtained by

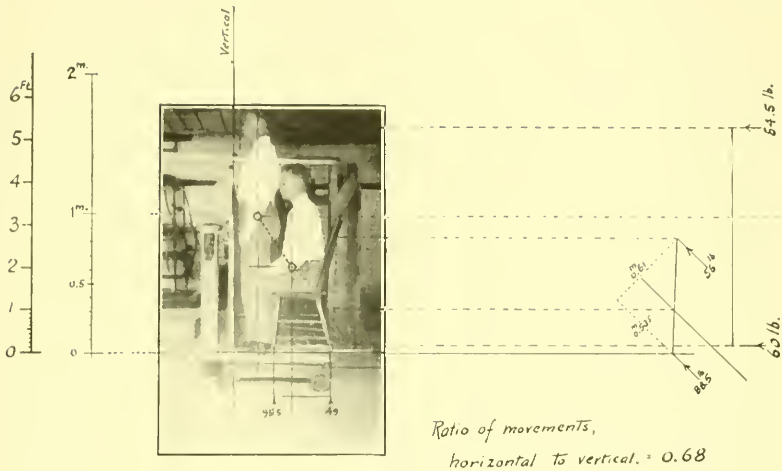
Vertical
6 Ft.
5
4
3
2
1
0
2 m.
1 m.
0.5
0
64.5 lb.
60 lb.
56 lb.
88.5 lb.
0.61 m.
0.535 m.
95.5
49
Ratio of movements,
horizontal to vertical. = 0.68

FIG. 1

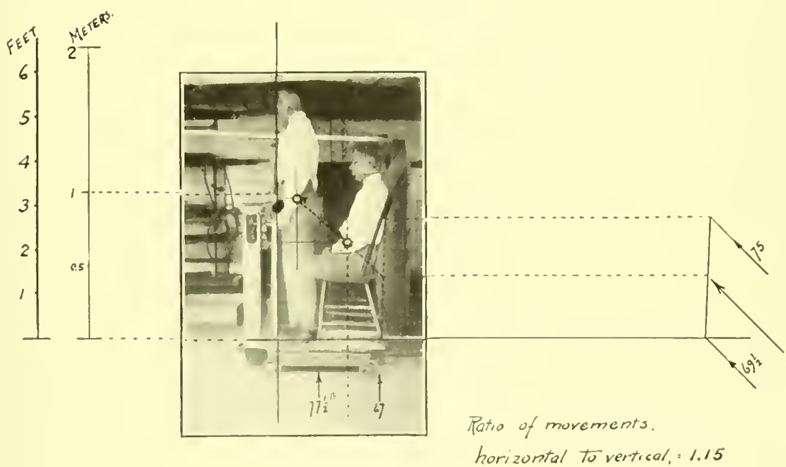
FEET
METERS
2
6
5
4
3
2
1
1
0.5
75
69½
77½ lb.
67
Ratio of movements.
horizontal to vertical, = 1.15

FIG. 2.

studying the movement of the center of gravity of the subject. (Plate XLI, Figs. 1 and 2.) If the ratio of the movements, horizontal to vertical, is taken as measuring approximately the relative intensities of the respective forces, it gives a fairly good indication of the backward shove exerted by a man when he jumps up to cheer a spectacular play on the football field, or to shout welcome to his political favorite. In Plate XLI, Fig. 1, the movement shown is from an erect sitting posture to a standing position, while Plate XLI, Fig. 2, shows a similar movement from a less alert sitting position. The first movement is probably more nearly typical than the second, but, in jumping up suddenly, as in Plate XLI, Fig. 1, in his final standing position a man would probably lean forward more than that shown in the photograph, so that a ratio of horizontal to vertical motion of about 0.75 might fairly be assumed. Although this ratio of movements may not show the relative force values exactly, owing to complexities of motion not taken into account in the simple geometry of the figure, it is undoubtedly fairly indicative. On such a basis, taking the increase in vertical effect at about two-thirds of the static weight, it appears that, in rising suddenly from his seat, a man may exert a backward horizontal shove about one-half as great as his weight, say from 70 to 80 lb. for an ordinary man.

Figs. 3, 4, and 5 are introduced to show the method of determining the centers of gravity of the standing and sitting figures. For the standing figure (Plate XLI, Figs. 1 and 2) the vertical centroidal line was readily drawn on the photograph by eye. The position of the horizontal centroidal line was then determined as in Fig. 3, by getting the reactions at head and heels on the two platform scales when the man was stretched out horizontally. A simple proportion between the two observed scale-weights then serves to fix the position of the line, and this is transferred to the photographs (Plate XLI, Figs. 1 and 2) by the construction shown at the right of the figures. Similarly, for the sitting figure, a vertical centroidal line was obtained by the method shown in Fig. 4, and an inclined centroidal line as in Fig. 5, the angle of inclination of the latter being determined by the two measurements, 24 in. horizontal and 21 in. vertical, shown in the sketch. The intersection of these two lines, when properly transferred to the sitting figure in the photograph (geometrical construction on the right), locates the center of gravity of the seated man. The scale for the photographs was fixed by the two meter sticks seen clearly in Plate XLI, Figs. 1 and 2, one laid

horizontally on the scale platform between the feet of the subject and the other standing vertically in the same plane. The vertical and horizontal components of the movement of the center of gravity were then measured directly on the photograph with this scale.

II.—Horizontal Forces Exerted by a Man Walking.—The motion of a man who is walking on a level floor, sidewalk, etc., at ordinary speed, is not perfectly uniform. The changes in velocity—whether acceleration or retardation—must be accompanied by some manifestation of horizontal force, this force urging the man forward when his velocity is being increased, and holding him back when it is being diminished. As these forces act on the man, his action (or reaction) on the floor must be equal and opposite to them; therefore, he will exert a backward horizontal push when his speed is increasing and a forward horizontal thrust when that speed is, for the moment, partly checked. The investigation which is summarized in Fig. 6 is an attempt to determine approximately what values these momentary forward and backward forces may have.

The twelve figures shown in the lower half of Fig. 6 were traced from a series of photographs, taken at intervals of $\frac{1}{12}$ sec., of an athlete walking.* The man walked directly in front of and parallel to a screen which was marked off by horizontal and vertical lines into squares of 5 cm. on a side. The subject was about 30 in. in front of this background, and the camera some 50 ft. from the subject, so that the distance interval covered in each interval of time might be determined with a close degree of approximation by counting the squares passed over by the man in each successive position. In making this count, the horizontal line, *AB*, was chosen as the base, as it passes very close to the center of gravity of the man for all positions, and the "reading" on the background squares was made at the intersection of this line with the buttocks of the naked figure. The total interval found in this way, from the first to the twelfth figure, was nearly 0.7 m., or about 5.5 ft., giving an average velocity of 6 ft. per sec. (a trifle more than 4 miles per hour) for the complete stride shown in the diagram. The average velocity during each interval ($\frac{1}{12}$ sec.) of

* This series of figures was traced from Plate 2 of Volume I of "Animal Locomotion," by Eadweard Muybridge (Philadelphia, 1887). This remarkable work, published in eleven great volumes, shows literally thousands of photographs of human beings and animals in great variety of motion. A smaller edition, containing a few of the series, much reduced in size, has been published under the title "The Human Figure in Motion." (London, Chapman and Hall, 1901.)

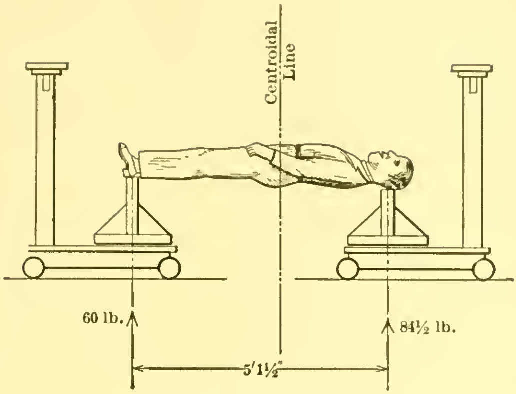

Fig. 3.

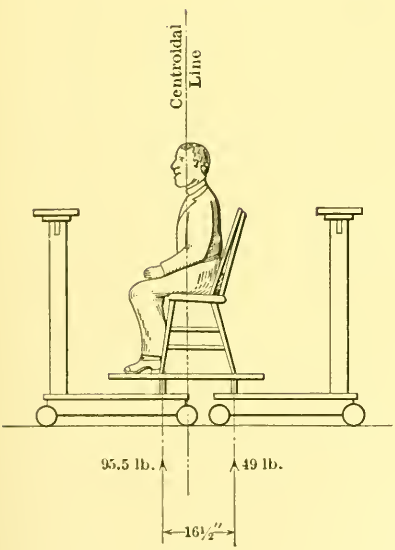

Fig. 4.

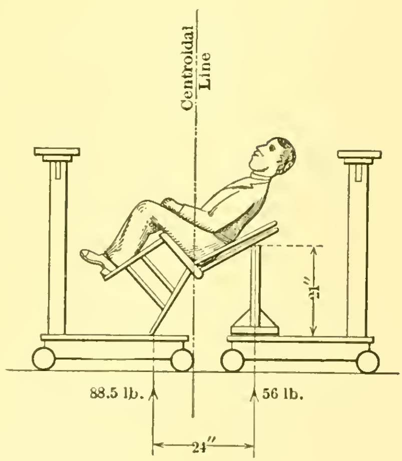

Fig. 5.

time may also be determined, and this has been done in the last column of Table 1.

TABLE 1.

Photographs.	Reading off background, in meters.	Distance Interval:		Velocity during interval, in feet per second.
		Meter.	Foot.	
1	0.18	0.18	0.59	7.1
2	0.36	0.14	0.46	5.5
3	0.50	0.12	0.39	4.7
4	0.62	0.15	0.49	5.9
5	0.77	0.15	0.49	5.9
6	0.92	0.20	0.65	7.8
7	1.12	0.16	0.53	6.4
8	1.28	0.14	0.46	5.5
9	1.42	0.13	0.43	5.2
10	1.55	0.13	0.43	5.2
11	1.68	0.17	0.56	6.7
12	1.85	....	....	....

In interpreting and applying these results, the uncertainties attendant on their computation must be kept in mind; they give, however, an interesting and instructive indication of the way in which the velocity varies. Thus, the velocity is well below the average between Positions 3 and 4, well above between Positions 6 and 7 and well below again between Positions 9 and 10. This is shown graphically in the upper curve of Fig. 6, which gives the velocity-time curve drawn as indicated by the computed values, not exactly through the plotted points but "smoothed out" to give an even, continuous curve. The average velocity for the stride is shown by the heavy dotted line.

To measure the force exerted, it is necessary to know the acceleration, and this is obtained by differentiating graphically the velocity curve. This acceleration curve, or "first-derived" of the velocity curve, is shown immediately below the latter. The ordinates of the acceleration curve were obtained by actual geometric construction for the slope of the tangent to the velocity curve at a number of points. It is seen that the high and low points of this acceleration curve have values, plus and minus, respectively, of 16 or 18 ft. per sec. per sec., which would correspond to a horizontal accelerating or retarding force of 80 or 90 lb. for a man weighing about 160 lb.* These force values urge the man forward for the high points of the curve, and backward for the low points; note, in this connection, the backward shove or

* Force (in pounds) = Mass $\times$ Acceleration, the mass being measured in "engineers' units" (1 unit = 32.2 lb.) and the acceleration in feet per second per second.

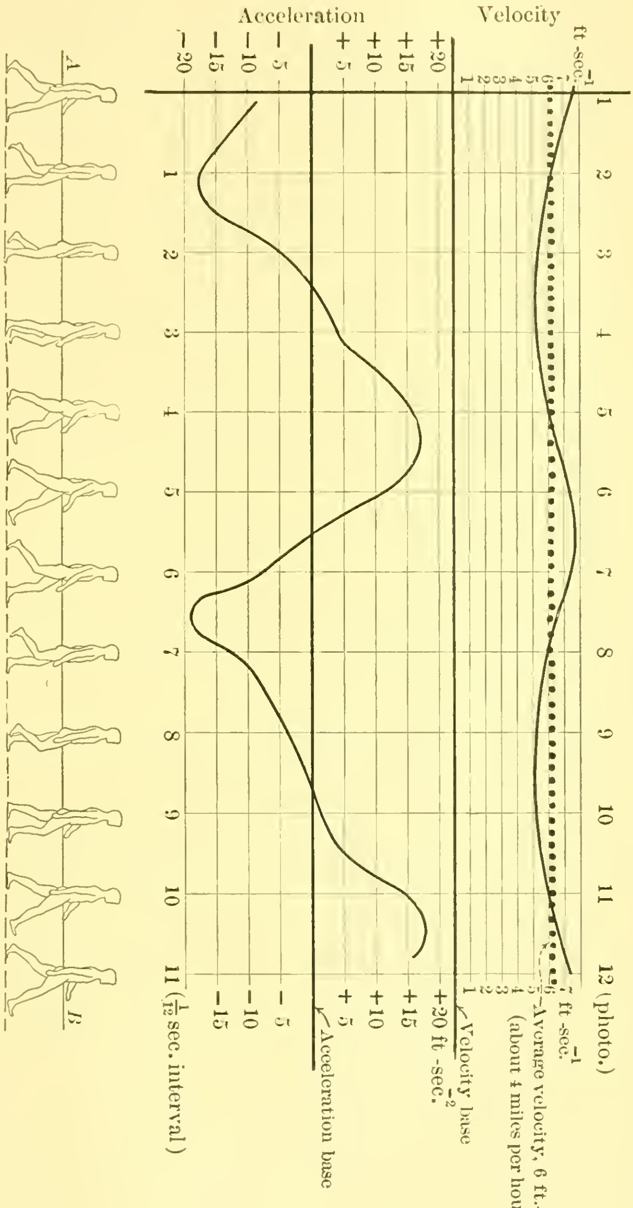
Acceleration
Velocity
ft.-sec.$^{-1}$
Average velocity, 6 ft.-sec.$^{-1}$
(about 4 miles per hour)
Velocity base
+20 ft.-sec.$^{-2}$
Acceleration base
12 (photo.)
11 ($\frac{1}{12}$ sec. interval)
A
B

Fig. 6.

kick of the right foot in Positions 5 and 6 and of the left foot in Positions 11 and 12, and the forward thrust, retarding the body's motion, of the right foot in Positions 1 and 2 and of the left foot in Positions 7 and 8. Although these forces are impulsive in their nature and act for only a fraction of a second, there seems to be no doubt of their existence, and although the amount is not determinable exactly (varying as it must with different individuals and conditions), it may, apparently, equal or even exceed half the static weight of the man.

III.—Horizontal Forces Exerted by a Man Running Across a Bridge.—Approaching from a slightly different viewpoint this problem of horizontal force exerted, some experimental data were collected bearing on the lateral effect which may be produced on a bridge. At the time of a boatrace, river pageant, or similar exhibition, a crowd of people will naturally gather on a bridge, ranging themselves along one side. As the spectacle moves under the bridge the entire crowd will cross to the other side, Plate XLII, and the motion is likely to be fairly rapid. What effect has it on the structure?

As in the previous cases, the experimenting was on an individual. The observer, with a stop-watch, noted the time of the subject as he turned from the rail on one side of the bridge and ran quickly to the other. This time included the start from rest at one rail and coming to rest again at the other. The suggestion was made that the subject imagine he was watching an exciting boat race, and did not want to miss any of it as the contestants pulled under the bridge. The tests were carried out on three bridges, in three different localities, the widths between rails being 17, 49½, and 68 ft., respectively, the second and third bridges having sidewalks on both sides. Three men took part as subjects, one a vigorous active man of sixty-seven, weighing 160 lb., another, thirty-nine years old, and weighing about 140 lb., and the third, twenty-six years old, and weighing 144 lb. The results are given in Table 2, and show, as might be expected, the higher average velocities for the wider bridges.

Remembering that these figures for the velocity are average values and not the maximum, we may obtain an approximation to the horizontal force exerted by the man, first as he started on his journey across the bridge and secondly when he stopped at the other side. Take, for example, the second bridge noted in Table 2 and the first

Crowd Rushing from One Side of a Bridge to the Other.

experiment. The maximum speed may be safely assumed as about 12 ft. per sec.—more, probably, rather than less—and the "mass" of the moving man at 5 units ($160 \div g$). The familiar expression for kinetic energy, $\frac{1}{2} M V^2$, then yields,

$$\tfrac{1}{2} \times 5 \times 144 = 360 \text{ ft-lb. of kinetic energy,}$$

and this energy must be given to the man at the start of his trip and destroyed at the end. If this is accomplished in the space of 2 or 3 ft. ($\frac{1}{5}$ to $\frac{1}{4}$ sec.), as seems reasonable, the impulsive starting, or stopping, force has a value of about 150 lb., nearly equal to the weight of the man. If it takes a longer time to attain the maximum velocity from the standing start—say, about $\frac{1}{2}$ sec.—the effort may be extended over a distance of 5 or 6 ft., and will amount to perhaps 60 or 70 lb. This effort, of course, must be exerted both at the start and at the finish, as the start is from rest and the subject comes to rest again at the opposite rail. If, as may happen when he brings up sharply against the rail, the whole amount of kinetic energy is destroyed in a space of 6 in. or 1 ft., the effort will amount to several hundred pounds.

TABLE 2.

Width of bridge, in feet.	Weight of subject, in pounds.	Time of running across bridge, in seconds.	Average velocity for one trip, in feet per second.
17	144	1.6 (down-stream)	10.6
		1.4 (up-stream)	12.1
		1.6 (down-stream)	10.6
		1.6 (up-stream)	10.6
		Average......	11 ft-sec.—[1]
17	140	1.4 (up-stream)	12.1
		1.6 (down-stream)	10.6
		1.8 (up-stream)	9.4
		1.6 (down-stream)	10.6
		Average......	10.7 ft-sec.—[1]
49.5	160	4.8 (up-stream)	10.3
		4.4 (down-stream)	11.2
		Average......	10.8 ft-sec.—[1]
49.5	140	4.0 (up-stream)	12.3
		4.4 (down-stream)	11.2
		Average......	11.8 ft-sec.—[1]
68	160	6.0 (up-stream)	11.3
		5.4 (down-stream)	12.6
		Average......	12 ft-sec.—[1]
68	140	5.2 (up-stream)	13.1
		4.8 (down-stream)	14.2
		Average......	13.7 ft-sec.—[1]

In applying these experimental results to determine the probable effect of crowds, several points must receive careful consideration. In the first place, the object of the whole inquiry is to get some sort of answer to the question: Against what loads, horizontal and vertical, should an engineer design a structure which is likely to have to carry a dense crowd of human beings? It is with respect to their bearing on this question that the experiment will be discussed.

Direct multiplication of individual effect by the probable number of units in the crowd is, of course, wrong. In the first place, the forces exerted are impulsive in their nature, being exerted for only a small, though finite and measurable fraction of a second. In order to get the full effect of such impulsive efforts from a crowd of people, it is necessary to have perfect synchronism of motion in every individual, a condition practically out of the question. Further, the denser a throng of people, the more individual motion is restricted, so that in the more closely packed crowds, giving the higher static loads per square foot, the increase resulting from kinetic effect is much reduced. On the other hand, that the static load only shall operate, perfect quiescence must be secured, a condition quite as impossible in any crowd as that of absolute synchronism in movement; and the duty of the engineer is to provide in every case for the maximum possible load-effect to which his structure may be subjected, a duty which is immeasurably emphasized when the safety of human beings is at stake.

The first experiment—that of rising from a crouching position—although suggestive and interesting, hardly has a practical bearing. That particular form of motion could take place only in a sparse crowd, and even then would be highly improbable. Its main value lies in showing rather strikingly the importance of some consideration of kinetic effect.

The case of a man rising suddenly from his seat, however, is of considerable importance. No one who has watched a grandstand full of euthusiastic football "fans" can doubt that a septacular play may bring nine-tenths of them to their feet with such a close approximation to unanimity of motion that the total kinetic effect must be considerable. However, if the usual allowance of 3 sq. ft. per sitting is made, and each spectator is assumed to weigh 165 lb., or 55 lb. per sq. ft. over the whole structure, an increase of 65 or 70% (over

the 55 lb. per sq. ft.) may be assumed without reaching the static value of 100 lb. per sq. ft. for which such a stand would probably be designed. Provision against horizontal effect, however, is not commonly made, and the importance of some such provision is illustrated by the experiment. To be on the safe side, a backward horizontal impulse of 70 or 80 lb. for each sitting might wisely be guarded against.

The "jouncing" movement, while it has a high kinetic intensity and is possible in a much denser throng than in Experiments I and II, is nevertheless of much shorter duration; the effect is that of a rather sharp, quick blow. On this account, practically perfect synchronism of movement is necessary to get the maximum effect, and this, of course, is quite impossible in any ordinary crowd.

The horizontal effect resulting from a man walking is probably not of general importance, except in the case of a large number of men marching in cadence, as a body of soldiers. The evil effect of this on bridges has been recognized for generations, and the tactical requirement of "breaking step" during the passage of a bridge by infantry is well known.* Mention might be made of the obvious slight variations in vertical load effect during the phases of a stride (in Fig. 6, it is seen that 4 and 9 are high positions and 1, 6, and 12 are low positions of the man's center of gravity), but these movements are small compared with those of Fig. 1 and Plate XLI, Fig. 1.

The experiment of running across a bridge is perhaps rather more difficult of application to a crowd. If a line of men, each weighing 150 lb., was distributed at intervals of, say, 18 in. along the railing of a bridge, and, at a given signal, turned and ran to the other side, bringing up sharply against the opposite rail, the lateral force exerted might even exceed the usual allowance made for wind. Something of the kind, undoubtedly, may occur under certain conditions, but to what extent it should be guarded against is again a matter of individual judgment based on the exigencies of a particular problem. A further application of this experiment, however, might be mentioned. A wharf or pier, used for excursion boats, may collect a large number of people who enter from the land singly or in groups and come to rest on the structure. In coming to rest a horizontal force is exerted, that is, kinetic energy is destroyed, tending to push the wharf out into

* See quotation from Robert Stevenson, cited previously.

the water; and this force is applied after the manner of a succession of blows, of varying magnitude, but all in the same direction.

The same is true of any elevated platform, or structure, built for the accommodation of men and women, the entrance to which is restricted to only one line of movement. Such a structure is bound to receive shocks or impulsive horizontal forces in the manner indicated. That these effects are generally so small as to be of no importance is quite true; but that they may also on occasion reach considerable proportions, and especially that the cumulative effect may be serious, seems to be equally beyond question. It may be that a bridge, or pier, or platform may be fully capable of carrying all ordinary loads, even to a densely packed crowd of people, but some day it gives way under a much less (static) load. Is it not possible that the failure may be due to a peculiar combination of movements, on the part of the individuals of the crowd, timed and synchronized so that a force effect is produced on the structure far greater than that of the mere dead load?

In conclusion, it may be said that the evidence submitted hardly warrants any dogmatic statement as to what loads should be considered in providing for the possible effects of motion in crowds of people. The results of the experiments are not scientifically exact, nor are the applications definite or precise; but that some provision should be made against effects which are shown to be probable in an excited gathering, there can be no doubt. It is a matter for the individual engineer to determine, in solving his particular problem. The facts, albeit with some uncertainties and many qualifications, are presented as they appeared in the actual experiments described. They should be interpreted and used in the light of sound common sense, fair and impartial engineering judgment, and, above all, a due sense of the responsibility that rests on the architect or engineer for the safety of those who must use his construction.

In closing, the writer would express his obligations to the friends and colleagues whose cordial assistance is recorded in Fig. 2 and in the tables. He is particularly indebted to his father, to whose virile pencil Fig. 1, Plate XLII, and part of Fig. 6, are due.

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PAPERS AND DISCUSSIONS

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THE ABSORPTION OF OXYGEN BY DE-AERATED WATER.*

By Earle B. Phelps, Assoc. Am. Soc. C. E.

The Report of the Metropolitan Sewerage Commission of New York, dated August 1st, 1912, and recently issued, contains, among other interesting material, a Report by Dr. W. E. Adeney, in which especial attention is given to the subject of re-aeration of polluted tidal waters. Dr. Adeney's views and conclusions are in many respects directly contradictory to those which William M. Black, M. Am. Soc. C. E., Colonel, Corps of Engineers, United States Army, and the writer reached after a study of this problem, in connection with New York Harbor, and fully stated in a report to the Board of Estimate and Apportionment of New York City, published March 23d, 1911. As this last-named report did not have a wide circulation, and as the matter under discussion is one of great importance, not only in the case of New York Harbor, but in all similar situations, it seems appropriate at this time to consider the facts and the conclusions that have been drawn therefrom by various workers.

Dr. Adeney made the first important contribution to this subject in his well-known experiments undertaken on behalf of the British Royal Commission on Sewage Disposal. His results are fully set forth in Appendix VI of the Fifth Report of that Commission.

He describes experiments, with glass tubes 6 ft. long and ¾ in. in internal diameter, in which he exposed de-aerated water under varying conditions, and, at the expiration of stated times, withdrew samples

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

from various depths and submitted them to analysis for dissolved oxygen and nitrogen. Both sea water and distilled water were used. He found that, with either kind of water, mere exposure of the upper surface to the air resulted in practically negligible re-aeration in the lower depths. He reported a tendency toward accumulation of the dissolved gases in the upper layers, which was greater in the distilled water than in the sea water, under these conditions. He next closed the top of the tube, introduced a small tube just below the surface of the water, and, by suction, caused a slight current of air to bubble through the top layer of water. This led to a slightly greater absorption on the part of the distilled water. In the case of the sea water, however, this method of procedure resulted in a fairly rapid absorption of oxygen and of nitrogen, and the quantities of these gases found at various depths at the conclusion of each experiment showed a tendency toward uniform distribution.

Dr. Adeney refers to this phenomenon as the "streaming effect", and identifies it with a similar effect obtained by Huefner under quite different circumstances. He adduces evidence, although inconclusive, that Huefner's theory of a direct streaming of the liquid, brought about by difference in specific gravity, is not tenable. He did not investigate fully the cause of the streaming effect, but believed it might be due to:

"* * * minute dust particles, or by other centres of condensation, possibly of an electrical nature, carried by the air current and being taken up by the water together with the gaseous constitutents of the air, and that these bring about in some way a sufficient density of the dissolved or condensed gases to render it possible for them to be drawn gravitationally downwards through water."*

The previous passage of the air through one tube of water destroyed to a large degree its power to bring about this streaming effect in a second tube, while the power was completely lost by passing it twice through water, thence into a third tube. Dr. Adeney suggested that the ions, or dust particles, or whatever may be the exciting cause, were removed by such treatment.

Col. Black and the writer showed by experiments with dyes and by determining the amount of evaporation resulting from the passage of dry or partly dry, air into a tube of water, following Dr. Adeney's

* British Royal Commission on Sewage Disposal, Fifth Report, Appendix VI, p. 61.

technique, that this streaming effect, so-called, was caused by a true streaming of water currents. These currents are caused by an increased specific gravity of the salt solution following evaporation of the water from the surface layer. This explains also the effect of using two or three tubes in series, and the absence of any material streaming in fresh water. In the latter case, a slight tendency toward streaming was observed, and, without further investigation, this was ascribed to a slight lowering of the temperature at the surface, due to evaporation, and the consequent increase in the density of the surface layer. Undoubtedly, also, the increased density resulting from the solution of atmospheric gases is a factor, as shown by Huefner. Although Dr. Adeney in his first report claimed to have disproven the theory of circulating currents of water, the explanation of this matter by Col. Black and the writer has been tacitly accepted by him in his report to the New York Commission, so that there need be no further discussion of the theory involved. The real point at issue is indicated in the following statements:*

"It is equally evident from these experiments that whatever may subsequently be discovered to be the true cause of the streaming, its effect in large volumes of sea or river water under natural conditions must be of great importance, and of such dimensions that the effect of ordinary diffusion may, in comparison, be entirely neglected."

Col. Black and the writer state:†

"In practice no such results would be observed, since the upper layers of an estuary are always somewhat fresher than the lower."

This latter statement, Dr. Adeney says, has no real foundation in fact. In this connection, reference to the New York Commission's report‡ will be of interest. Here is shown the condition of the water at the Narrows as regards both dissolved oxygen and salinity of percentage of sea water, at seven different stages of the complete tidal cycle. A distinct stratification is shown in each instance. From the surface to the 20-ft. depth there is an average increase of about 6% of sea water. In the case which shows the greatest uniformity throughout the depth, there is more than 2% increase. Moreover, the condition at the Narrows is less favorable to the contention of Col. Black and the writer than the majority of the cases shown in the

* British Royal Commission on Sewage Disposal. Fifth Report, Appendix VI, p. 64.

† Report of Col. W. M. Black and Prof. Earle B. Phelps concerning The Location of Sewer Outlets and The Discharge of Sewage into New York Harbor, 1911, p. 52.

‡ Metropolitan Sewerage Commission of New York, 1912, p. 119.

diagrams which follow. At the mouth of the Hudson, the gradient ranges from 6 to 8% of sea water in 20 ft., and in the upper Hudson, at Mount St. Vincent, it is normally more than 10% in the same vertical distance.

There seems to be, therefore, sufficient "real foundation in fact" for the assertion that stratification does exist, and that there is, therefore, no such vertical circulation of water from top to bottom, as Dr. Adeney found in his tubes, which were initially at a uniform concentration. If further foundation in fact be required for this almost self-evident proposition, reference may be had to the work of Morton F. Sanborn, Assoc. M. Am. Soc. C. E., on "The Distribution of Sea Water in the Charles River Basin."* As a result of a year's study, with almost daily examination of the vertical distribution of sea water in various parts of the Charles River Basin at Boston, Mr. Sanborn found stratification to be so persistent that the most severe storms produced only a temporary uniformity, even in the upper 20 ft. of the water, and that, after the cessation of such storms, stratification was re-established. The analytical evidence is direct and positive, and rests on no theory. The streaming effect which depends on the presence of more dense layers above those of less density, is impossible in waters that show increasing concentration of salt with increasing depths. These conditions exist in the cases that have been cited; therefore, the phenomenon of streaming does not and cannot occur.

Dr. Adeney's further suggestion that the sewage matters are concentrated in the upper layers is obviously true, but beside the question. We are considering the rate of absorption of dissolved oxygen by a polluted water, under the restriction that the dissolved oxygen content shall not be reduced below a certain specified amount. This value, if properly determined, will limit the amount of sewage which may be received into the water in question, under the restrictions imposed. The most favorable assumption that can be made is that the sewage is uniformly distributed. This, in other words, will give a maximum permissible pollution. If, on the other hand, we assume a concentration of pollution in the upper layers, then either the amount of permissible pollution will be decreased in proportion to this decreased volume of water available, or the conditions at the surface will be reduced below the assumed standard. Such a concentration of

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* *Engineering News,* March 10th, 1910, p. 272.

sewage near the surface would result in a greater average rate of oxidation than if there were uniform distribution of the same quantity of sewage, but it is not the condition of maximum re-aeration.

Dr. Adeney next refers to the experimental technique by which Col. Black and the writer determined the diffusion coefficient. He first points out that that coefficient bears no resemblance to the one determined by Huefner, and believes that our series of ascending values with increasing temperature are due to a streaming effect, because of failure to take extreme precaution in preserving the purity of the water used. With regard to the first point, the apparent discrepancy is merely one of units used in the formula. The exact definition of a complicated coefficient of this kind cannot be simply made, and the reader is referred to the original paper (page 89) for the definition of the coefficient and the units in which it is stated. It would be a laborious matter to convert Huefner's coefficient to these units, and the magnitude of the experimental work was so much greater than his, and, in the writer's estimation, the method of experimenting was so much more reliable, that no attempt has been made to recompute the results in other terms. The form in which Col. Black and the writer have derived it is most favorable to practical application to the problem in hand.

As for the general accuracy of the analytical work, the results must speak for themselves. It would seem that Dr. Adeney's statement, "That the results obtained by these observers do not represent the true co-efficient of diffusion of oxygen in water becomes evident on carefully examining them" (page 83), and his further statement that these results have "no resemblance nor relationship to the true co-efficient," might have been accompanied with more explanatory matter for the benefit of those less familiar with the problem. Col. Black and the writer in their report give a plot, Fig. 1, showing the results of 114 tests made throughout a considerable range of temperature. Dr. Adeney quotes from page 94 of that report as follows:

"The values are somewhat scattering, and a closer analysis indicates that some unknown factor is involved. Exceedingly concordant results are readily obtained upon any one day, while upon the following day another group of concordant results will be obtained differing from the first. Barometric corrections were later applied without relieving the situation."

From this statement, Dr. Adeney concludes that our values have been arbitrarily selected, and will give misleading results. The plot, Fig. 1, shows the general character of the errors that have been referred to. There are admittedly imperfections in the process, and the

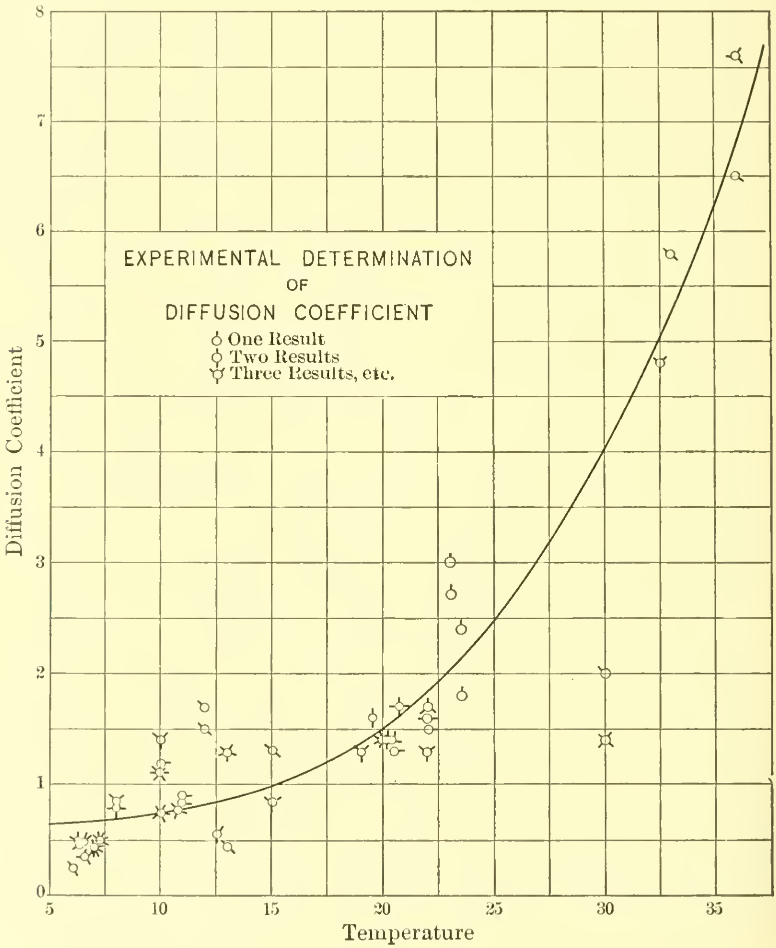

Fig. 1.

results are not of extreme scientific accuracy. For this reason, the work, which was originally carried out as a contribution to pure science, was not published as such. On the other hand, as stated on page 95:

"Within the range of values which have been employed in the computations in the text, errors resulting from the maximum divergence of individual tests from the average value of the curve are less than 5 per cent. at the five-foot depths and less than 2 per cent. at the twenty-foot depths."

Surely this is sufficient accuracy for an engineering discussion involving so many unknown factors, as does the New York Harbor problem. The increasing value of the coefficient with increasing temperature, the second point objected to by Dr. Adeney, was not made use of in our discussions. It may be noted, however, that this change is thoroughly concordant, both in direction and in amount, with diffusion coefficients which have been determined for various soluble salts in water. With higher temperature, the molecular velocity of a dissolved substance (that is its tendency to diffuse) increases, and the viscosity of the solution (the resistance to diffusion) decreases.

Dr. Adeney gives (page 85) the results of an experiment from which he concludes: "* * * whether evaporation can or cannot occur, Black and Phelps' proposed formula yields seriously misleading results." The details of the experiment as published are not sufficient to enable one to check our formula by definite computations The experiment is apparently made with "air", and the results are referred to as "Degree of Aeration." The portions of the experimental tube that were examined are defined as the upper, middle, and bottom layers, without any definition of the extent or position of these layers. It is impossible, therefore, to prove or disprove the statement quoted above. Comparison of these experiments with Experiments 8 and 9, reported on pages 62 and 63 of Dr. Adeney's original Report to the Royal Sewerage Commission shows the two sets of experiments to be identical in all their essential features, and, in all probability, the New York experiments numbered 3 and 4 are merely recomputed from the original work. If this supposition is correct, Dr. Adeney has made seven errors in attempting to determine the accuracy of our formula by comparison with these results:

(1) The formula is misinterpreted in that it is assumed to give the increased oxygen content at any depth, whereas its derivation and our use of it show that it gives the total oxygen absorbed by a column of stated depth. This is plainly stated on page 91 of our report.

(2) Our 20° coefficient is used, although the temperature of the

experiments is about 8.5 degrees.* Our coefficient for that temperature is just half the coefficient at 20 degrees.

(3) The "re-aeration" was measured by nitrogen, while our coefficient refers to oxygen. There is no evidence that the two gases behave similarly.

(4) The experiments are in sea water, whereas our coefficient refers to fresh water, and we state explicitly that the sea water coefficient is necessarily lower, owing to the increased viscosity of sea water. This effect would be exaggerated at low temperatures.

(5) The computed value was based upon the supposition that the water was free from oxygen at the start, whereas it contained more than 16% in each experiment. Proper use of our formula takes account of the original oxygen content, a self-evident matter which Dr. Adeney consistently overlooks.

(6) In recomputing his results to the basis of "per cent. saturation," the temperature of the room, 10° to 13.5°, is used, instead of the temperature of the water jacket, 8.5 degrees.

(7) In these recomputations, values of zero are arrived at in the middle and bottom layers, whereas in each case in the original experiments a definite amount of re-aeration was noted. The quantities are small, but too large to be ignored when the argument is based upon zero values.

If the writer is in error in assuming that these two sets of results refer to the same experiments, it is merely necessary to point out that they are obtained under identical conditions, so far as the conditions are stated, and, in any event, the writer will deal with the original work, which is fully described, rather than with the apparently recomputed values of the New York Report. The figures of the original report, of the New York Report, and of the former recomputed to the form of the latter, are given in Tables 1 and 2.

The columns referred to in Tables 1 and 2 are the long glass tubes. Air was bubbled through the top layers of three of these in series. The results of the first columns are not given, for the circulating currents established vitiated the diffusion effect.

The experiments are not capable of giving an accurate measure of the diffusion coefficient because of the large percentage errors that are possible and evident in the lower depths of all the columns. Neither are they planned to take advantage of our formula, which gives the

TABLE 1.—DR. ADENEY'S EXPERIMENTS WITH SEA WATER. Original Experiments. Royal Sewerage Commission Report, Pages 62-63. Amount of nitrogen in cubic centimeters per liter.

No.	Time, in hours.	Temperature, Centigrade.		1 to 200 mm.		800 to 1 000 mm.		1 600 to 1 800 mm.	
				Columns:		Columns:		Columns:	
		Room.	Jacket.	2	3	2	3	2	3
8.....	Start.	12.8° to 10.0°	8.6° to 8.3°	2.13	2.51	2.13	2.51	2.13	2.51
	48			11.15	11.09	2.29	2.55	2.18	2.54
9.....	Start.	13.7° to 13.5°	8.4° to 8.5°	1.63	1.51	1.63	1.51	1.63	1.51
	48			7.15	6.93	1.65	1.56	1.65	1.52

TABLE 2.—DEGREE OF AERATION, AS PERCENTAGE OF SATURATION.

No.	Time, in hours.	Temperature, centigrade.	Uppermost Layer.		Middle Layer.		Bottommost Layer.	
			Columns:		Columns:		Columns:	
			2	3	2	3	2	3
		Computations in New York Report, Page 85.						
3.....	48	10.0	72	69	0	0	0	0
4.....	48	13.5	47	47	0	0	0	0
		Computations by the Writer for Nitrogen at 8.5 Degrees.						
8.....	Increase in N in 48 hours (in cubic centimeters)		9.02	8.58	0.10	0.04	0.05	0.03
	" " " (percentage of saturation)........		70	67	1.24	0.31	0.39	0.23
9.....	Increase in N in 48 hours (in cubic centimeters)		5.52	5.42	0.02	0.05	0.02	0.01
	" " " (percentage of saturation)........		43	42	0.16	0.39	0.16	0.08

total oxygen absorbed in the entire column. In the course of establishing the formula, we derived an expression for the change in oxygen concentration at any depth. This is shown on page 92, just before the integration. While we have no determination of the coefficient for nitrogen in sea water, it may not be without interest to note that by this formula, under the stated conditions of depth, temperature, and initial content of the water, the increase in oxygen in a column of fresh water at the bottom layer, depth, 1 700 mm., would amount to but 0.1% of saturation, instead of 6%, which Dr. Adeney computes. In Experiment 9, in which the temperature was maintained constant

within 0.1°, the results were 0.16% and 0.08% in the two tables. In the other experiment the temperature varied 0.3°, and the results were 0.39% and 0.23%, respectively. Undoubtedly, all results are high, owing to changing temperature and circulation in the tubes. It is evident, however, that the diffusion coefficient of nitrogen in sea water is somewhat lower than that of oxygen in fresh water, and that the mathematical analysis of this problem by Col. Black and the writer, if not supported, is at least not invalidated by Dr. Adeney's figures.

Discussions of theory and of laboratory investigations on such a complex problem as this are likely to be futile, unless the results can be controlled and checked against actual conditions. The problem of re-aeration is an enormously complicated one, but, fortunately, New York Harbor provides a full-scale experiment, and the various investigations which have been made on its waters have furnished a considerable mass of analytical and other data. It will be of value, therefore, to inquire to what extent Dr. Adeney's values for re-aeration in a harbor water as a whole differ from those of Col. Black and the writer, and if there be serious difference, how the two sets of values compare with existing conditions. Dr. Adeney states* that any pollution which would not cause a greater rate of absorption than 0.055 c.c. of oxygen per liter per hour would exercise no apparent effect on the aeration of a tidal water. The context shows that he means the condition of aeration, not the rate of re-aeration. Col. Black and the writer have already discussed the errors of this point of view, which gives no consideration whatever to the initial conditions of aeration or of depth. It must be obvious that a saturated water will absorb no oxygen; that there must be a reduction from the normal condition of aeration before absorption begins; and that, furthermore, the greater this reduction the greater the absorption. The figure quoted refers to a mixture of salt and fresh waters in equal proportions, and the value for a mixture containing 75% of sea water is 25% greater. This is the statement taken literally and without question by Edlow W. Harrison, M. Am. Soc. C. E., upon which he based his computation cited by Col. Black and the writer (page 49) to show that the waters of New York Harbor could provide for the sewage of 60 000 000 people without reducing the normal oxygen. Dr. Adeney, in his New York report, makes no attempt to treat the question quantitatively, but points out the serious

* British Royal Commission on Sewage Disposal, Fifth Report, Appendix VI, p. 67.

variation in important factors, and believes that the rate of re-aeration may vary from zero to such a magnitude as to be of practical importance. His only approach to a quantitative statement is to the effect that rates of 0.067 to 0.077 c.c. of oxygen per liter per hour are probably maxima. The average of these two values is equivalent to 0.86 lb. per million gallons per hour, a figure which will be used later for comparison. This value refers to 6-ft. depths, and Dr. Adeney concludes that in greater depths the rate will vary inversely, so that in 12-ft. depths we may assign 0.43 lb. as Dr. Adeney's maximum. In Dr. Adeney's original report, 6-ft. columns were taken to represent actual conditions in practice. Recognition of the depth factor is apparently due to the Black and Phelps formula. Col. Black and the writer also realized the complex nature of the problem, but undertook a definite solution which would involve the most important variables. They estimated average rates of absorption, in 12-ft. depths, of about 0.045 lb. per million gallons per hour, although this figure varies considerably in the various portions of the bay.

Comparison of these figures with Dr. Adeney's maximum, after modification for depth, shows them to be about one-tenth as great. Dr. Adeney confesses his inability to determine an average rate, but claims that it is "of such magnitude as to be of great practical importance." Our figures are average rates, and represent an immaterial quantity in the treatment of the whole subject. While Mr. Harrison's estimate would probably be an unfair statement of Dr. Adeney's views, as now modified in his New York report, it should be noted that it is the only computation that has ever been made to show the value of the "streaming effect," and Dr. Adeney has made no attempt to correct it, or to replace it with a more conservative computation. The Black and Phelps formula has been applied to the conditions existing in New York Harbor, and, in conjunction with Col. Black's excellent mathematical analyses of the tidal conditions existing, has been used as a basis for a computation of the actual condition of the waters of various parts of the harbor. Instead of providing for 60 000 000 people without reduction of the normal oxygen of the waters, we find the effect of re-aeration to be practically negligible. If there be no reduction from the normal there will be no re-aeration. If 30% of the normal oxygen of the harbor waters be used up in sewage oxidation, so that the waters stand at 70% of saturation, the additional aeration

will amount to less than 1% of saturation in 12 hours. Under these conditions, oxygen obtained from the atmosphere by absorption would care for the sewage of about 250 000 people, out of a total of 7.4 millions which are provided for by the 30 per cent. The fact that, with less than this population contributing sewage to-day, the waters of the harbor do not stand at their normal oxygen content, but are on the average already reduced to approximately 70% of saturation, is of interest. The calculated condition, assuming practically no re-aeration, agrees closely with the facts. A more severe test of our conclusions, however, is possible. The Metropolitan Sewerage Commission made 1 870 analyses for dissolved oxygen in and near New York Harbor during 1911. This work was confined mainly to typical cross-sections. Table 3 shows the original computations by Col. Black and the writer, in comparison with a summary of the Metropolitan Commission's results.

TABLE 3.—DISSOLVED OXYGEN IN NEW YORK HARBOR WATERS.

A. Calculated by Black and Phelps, February, 1911.

B. Analyses by Metropolitan Sewerage Commission for the Year 1911, Reported August 1st, 1912.

Location.	Percentage of saturation.	
	A	*B*
Lower Bay.		
End of flood	100	100
End of ebb	71	74
Upper Bay.		
End of flood	76	79
End of ebb	54	55-56
Hudson to 96th Street.		
End of flood	54	56
End of ebb	48	54
East River.		
Mouth to Hell Gate.		
End of flood	53	55
End of ebb	71	57
Hell Gate to Throg's Neck.		
End of flood	71	57
End of ebb	81	86

The computations refer to the areas indicated in Table 3, while the Commission's figures are necessarily for cross-sections. In each case the value for the area in question at the end of the flood is compared

with the average figures for flood and ebb at the cross-section down stream from the area in question; and at the end of the ebb with similar values for the cross-sections up stream from the area in question. In the single case of the Hudson River at 96th Street at the end of the ebb there are no up-stream data available. The figure given is the average value for the ebb flow at the mouth of the Hudson. Obviously, the character of the water remaining in this section at the end of the ebb is worse than the average ebb flow, a fact which is indicated in the discrepancy of these two figures. With a single exception, the remaining figures show a striking agreement. There is evidently something wrong with our value at the region of Hell Gate, which has influenced the computed value of the ebb-waters below Hell Gate and the flood-waters above. It might be suggested that there is an influence of the Harlem River shown here which was not properly taken into account in our computation.

The agreement between the computed and the actual conditions, although gratifying, is of itself of little practical moment. Its value lies in the fact that it demonstrates the substantial accuracy of the methods which have been used in the analysis of this problem, and makes it possible to use this same method in computing the probable effect of further increase in population, or of the discharge of additional quantities of sewage from regions not now tributary to New York Harbor. All the facts seem to support the view that aeration in New York Harbor is an immaterial factor in the whole problem of sewage disposal.

Upon the grounds of theory, experiment, and experience, therefore, the writer, much to his personal regret, finds himself obliged to differ from Dr. Adeney, whose personal acquaintance he has the honor to enjoy and for whose scientific work he has the most cordial regard.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

SOME EXPERIMENTS WITH MORTARS AND CONCRETES MIXED WITH ASPHALTIC OILS.

By Messrs. Arthur Taylor and Thomas Sanborn.

To be Presented April 16th, 1913.

Object of Tests.

Recently, the attention of engineers has been called to some tests of oil-mixed mortar and concrete,* by Logan Waller Page, M. Am. Soc. C. E., which are held to indicate that oil will mix uniformly with concrete or cement mortar, forming what may be termed an emulsion, and that concretes and mortars, thus mixed with certain percentages of oil, are "absolutely water-tight under pressures as high as 40 lb. per sq. in." and, "under low pressures, both are waterproof."

In order to determine whether these results would hold true for concretes mixed with western (asphaltic) oils, the writers began a series of tests in February, 1912, laying special emphasis on the question of permeability under pressure.

Conclusions.

1.—The following experiments, although incomplete and inconclusive, indicate that, with western (asphaltic) oils, the facts stated by Mr.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* "Some of the Properties of Oil-Mixed Portland Cement Mortar and Concrete," *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 255.

Page in regard to strength and water-tightness of oil-mixed concretes and mortars are open to serious question, and should not be accepted in practice until further experiments have demonstrated their truth more fully.

2.—Concrete and cement mortars with 5, 10, and 15% of oil, by weight of cement, were found by these tests to be more permeable than concrete and cement mortars without oil, when subjected to pressures of 20, 40, and 60 lb. per sq. in.

3.—The strength of mortars was found to be affected seriously by tho addition of as low as 5% of oil.

4.—The absorption of water by mortars under low heads was found to be but little decreased by the addition of light oils, and increased with heavy oils.

Description of Tests.

Great care was taken to secure uniformity of qualities and quantities in all the experiments.

In the permeability tests, three specimens of each mixture were tested; and in the tensile and compressive tests, either three or six specimens of each kind were used.

The quantity of oil used in each case was determined as a percentage, by weight, of the quantity of cement. It was found, as stated by Mr. Page, that the western (asphaltic) oils will also readily emulsify with wet cement, cement mortar, or concrete, and can become mixed so thoroughly in the mass that on examination with a reading glass no unassimilated globules of oil are visible. The mixture has a rich glossy appearance, the oil, apparently, has spread itself out in a thin film enveloping each particle of sand and rock. Even a very thick oil—one which is so viscous that when cold it cannot be poured—will mix and disappear in concrete or mortar when it is heated and mixed with hot aggregates. The amount of mixing was found to be a principal factor in determining whether or not the oil is incorporated completely and uniformly with the other ingredients of a mortar or concrete.

The consistency of the concrete or mortar also governs the degree of emulsification; the wetter the mixture the better the oil emulsifies.

Materials.

The oils used were commercial products having trade names as indicated. Their mechanical analysis is shown in Table 1.

TABLE 1.—Mechanical Analysis of Oils.

Name.	Baumé gravity.	Flash point.	Burns.	Asphaltum.
Boiler Fuel	17°	171° Fahr.	....	42%
Richmond Road Oil No. 6	13°	260° "	400° Fahr.	75%
Richmond Fuel	16°	210° "	300° "	52%
Star Fuel	24°	195° "	245° "	21%
Liquid Asphalt	0°.9	....	....	92%

Portland cement complying with standard specifications, clean river sand, and two kinds of river gravel were used for the concretes.

One gravel was graded from 1 in. to ¼ in., the other from ¼ in. down. The percentages of voids were: Sand, 33; fine gravel, 37½; coarse gravel, 49. Portland cement and standard sand were used in all mortars. The proportions used for concrete were for an aggregate of maximum density. The mechanical analysis of the aggregate is shown in Table 2.

TABLE 2.—Mechanical Analysis of Aggregates.

Diameter, in inches.	Percentage passing through sieve.
1.00	100
0.50	72
0.25	57
0.19	52
0.06	41
0.03	32
0.022	25
0.012	15

Methods of Mixing.

Concretes.—The proportions of aggregate are by volumes, loose. The quantity of oil is given as a percentage of the weight of the cement. The quantity of water giving a quaky mixture was determined and used thereafter.

All mixing was done in a box having an approximate volume of 2 cu. ft.

The sand, fine gravel, and cement were mixed in the usual way, and the oil was poured on the wet mass and thoroughly mixed in until it had completely disappeared, as shown by uniform color and absence of oil spots. To this mixture the previously soaked coarse gravel was

then added, and the whole was mixed again. In order to insure uniformity of mixing, the time of each separate operation was fixed and thereafter followed according to schedule.

The most viscous oils, namely, liquid asphalt and road oils, required a larger percentage of water (for the same consistency) than the mixtures using the lighter oils.

Mortars.—The proportioning of mortars was done by weight. Three parts by weight of standard (Ottawa) sand and one part of cement were mixed to normal consistency; the oil was then poured on the mortar, and the whole was mixed until the oil disappeared and the resulting mass was of uniform color. The viscous oils required more mixing than the lighter oils to bring about this condition.

Neat Cements.—Standard methods of mixing the cement paste were used, the oil being poured on the wet paste and mixed as before until the mass was of uniform color. Apparently there is no limit to the quantity of oil that may be used, as far as emulsification is concerned, the cements containing the largest percentages of oil emulsifying as readily as those containing smaller quantities. The oil and wet cement mix together as perfectly as two grades of oil.

Methods of Testing.

For tension and compression tests, the usual standard methods were used. The permeability tests were made with apparatus similar to that used for such tests at the University of Wisconsin. The apparatus consists of four lengths of $\frac{3}{4}$-in. pipe tapped into a horizontal main. Each of these pipes connects with an 18-in., glass water-gauge fitted with a calibrated scale for measuring the quantity of water between certain heights shown on the gauge. On the end of each vertical pipe there is a union for connecting the specimen holder. These holders are of 6-in., threaded, wrought-iron pipe, 11 in. long, fitted with cast-iron caps which screw down on a rubber gasket, thus making a water-tight joint. A $\frac{3}{4}$-in. pipe from this cap connects with the union previously mentioned.

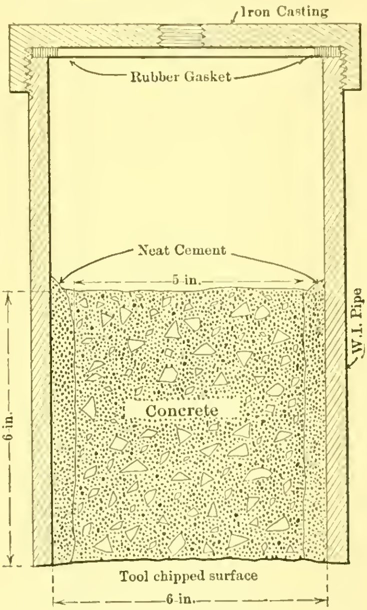
After the specimens are connected up, water is let in from the top, through a valve on the horizontal pipe, until it stands at a certain height on the gauge glass. The pressure is applied by compressed air, which is kept constant by two supply tanks and is gauged at its entrance to the apparatus.

A valve is placed between each specimen and the horizontal main, so that a leaky or broken specimen can be shut off without disturbing the test of the other specimens.

The permeability specimens were made in the holders previously mentioned, Fig. 1. The concrete specimens were 6 in., and the mortar specimens 3 in., thick. Before the concrete or mortars were put in place, a ½-in. lining of neat cement was plastered on the inside of the pipe, which insured a perfectly tight bond.

All specimens stood dry for 24 hours after mixing; water was then put on top of the specimen until the test was made.

The top and bottom surfaces of each specimen, before being attached to the testing machine, were chipped uniformly by hand tools, in order to provide for each specimen a surface as nearly uniform as possible.



CROSS SECTION OF PERMEABILITY TEST SPECIMEN

Fig. 1.

Results of Tests.

Time of Set.—It was found that, with oil-mixed specimens, the time of setting was retarded, in both the initial and final set. These tests were made with the Vicat needle according to the Standard Specifications. The oil used was an average middle-grade oil. Table 3 gives the time of initial and final set for various percentages of oil.

TABLE 3.—Time of Initial and Final Set.

Percentages of oil.	Initial set, in hours.	Final set, in hours.
0	4.	5.20
5	5.20	6.
10	5.80	7.30
15	6.80	9.50
25	12.20	36.

These results confirm the observations of Mr. Page.* A theory may be advanced that the time of setting is retarded and the strength is decreased, due to the fact that the oil film separates particles of cement from each other, so that there is not the complete intimate association of every particle of cement, as there is in a mixture without oil.

Tensile Strength.—Either three or six specimens of each kind were broken. All tests were made on specimens set 24 hours in moist air, the remainder of the time under water. Table 4 gives the strength and relative values of specimens having various percentages of oil.

TABLE 4.—Tensile Strength of 1 : 3 Mortar.

Averages of three or more briquettes.

Percentage of oil.	7 Days.		28 Days.	
	Pounds per square inch.	Relative strength.	Pounds per square inch.	Relative strength.
		Percentage.		Percentage.
0....................	324	100	430	100
5 Boiler Fuel.......	260	80.3	335	78
10 "	192	59.3	303	70.5
15 "	170	52.5	225	52.4
25 "	137	42.3	213	49.6
Richmond Fuel, 10..	170	52.5	273	63.5
Road No. 6, 10.......	146	45.0	184	42.8
Liquid Asphalt, 10..	141	43.5	220	51.2

Crushing Strength.—All compressive specimens were made of 1 : 3 mortar in 3-in. cubes. Specimens stood 24 hours in moist air; the remainder of the time before testing they were under water. All tests were made on an electrically-driven machine, the load being applied slowly. In every test the specimens failed regularly, the sides sloughing off uniformly. In Table 5, are given the compressive strengths and relative values of oil-mixed specimens compared with plain or no-oil mortar.

Absorption.—Mortars containing percentages of middle-grade oil up to 25 were slightly less absorptive than plain mortars, the absorption decreasing with the increase in the percentage of oil. The specimens containing heavier oils showed an increase in absorption.

* *Loc. cit.*, p. 258.

It was found that at the end of 24 hours the mortars had reached just about complete absorption. At the end of 72 hours the weight had only increased a negligible fraction over that at the end of 24 hours.

TABLE 5.—28 AND 50-DAY COMPRESSION TESTS: 3-INCH CUBES OF 1:3 MORTAR.

Percentage of oil.	28 DAYS.		50 DAYS.	
	Pounds per square inch.	Relative value.	Pounds per square inch.	Relative value.
		Percentage.		Percentage.
0....................	3 950	100	4 400	100
5 Boiler Fuel.......	2 435	61.6	3 620	82.4
10 "	1 780	45.1	2 460	56
15 "	1 460	37.0	2 000	45.5
25 "	712	18.2	1 000	22.8
Richmond Fuel. 10..	1 640	41.5	.....	.....
Road Oil No. 6, 10...	1 235	31.2	.....	.....
Liquid Asphalt, 10..	1 080	27.4	.....	.....

Table 6 gives the results of the absorption tests.

TABLE 6.—RESULTS OF ABSORPTION TESTS, WITH 1:3 MORTAR, 35 DAYS OLD.

	Boiler fuel oil.					Richmond Fuel.	Road Oil No. 6.	Liquid Asphalt.
Percentage of oil, weight in grammes....	0	5	10	15	25	10	10	10
Weight after heating......................	140	131.7	142	135	134	199	204	139
Weight after 24 hours in water..........	143.5	134.3	145.2	137.5	136	205.8	110.5	144.5
Weight after 72 hours in water..........	negligible gain over 24 hours immersion.							
Gain in weight in 24 hours..............	3.5	2.5	3.2	2.5	2	6.8	6.5	5.5
Percentage of gain in water in 24 hours..	2.5	1.97	2.26	1.85	1.49	3.42	3.19	3.96

Permeability Tests.—The permeability tests were made with the specimens heretofore described; at the end of 20 days with concrete, and 10 days with mortar. Three of each were tested, and the average value was taken, except in some cases when the specimens proved to be faulty. Water pressure was applied on concrete for periods of 4 hours each under pressures of 20, 40, and 60 lb. per sq. in., and finally a 4-day test under 15 lb. per sq. in. was run. With mortar specimens, 10- and 20-lb. pressures were applied for 4 hours under each pressure. Tables 7, 8, and 9 give the number of cubic centimeters of water passed through the specimens under the various pressures.

TABLE 7.—PERMEABILITY TESTS OF 1:3:4 CONCRETE.
All specimens under pressure for 4 hours.

Set.	Kind of oil.	Percentage of oil.	CUBIC CENTIMETERS OF WATER PASSED THROUGH.		
			20 lb. per sq. in.	40 lb. per sq. in.	60 lb. per sq. in.
*A**.....	None........................	0	64	80	83
A......	Boiler Fuel..................	5	137	114	108
A......	" "	10	119	72	53
A......	" "	15	66	62	38
B......	None........................	0	32	41	60
B.....	Boiler Fuel..................	5	143	128	160
B......	" "	10	68	65	....
B......	" "	15	87	119	150
C.......	None........................	0	40	41	105
C.......	Boiler Fuel..................	5	42	53	150
C.......	" "	10	47	43	110
C.......	" "	15	38	40	108
AVERAGE OF *B* AND *C*.†					
	Boiler Fuel..................	0	36	41	82
	" "	5	42	53	155
	" "	10	57	54	95
		15	62	79	129
D......	None........................	0	10	12	12
D.....	"	0	9	10	10
D......	"	0	11	10	9
E......	Richmond Fuel.............	10	24	31	41
E......	" "	10	20	25	28
E.....	" "	10	29	37	44
F......	Road Oil No. 6..............	10	68	85	75
F......	" "	10	56	70	53
G......	Liquid Asphalt.............	10	22	23	33
G......	" "	10	21	23	32
G......	" "	10	23	25	38

* Set *A* stood dry 24 hours before testing. As the results for increasing pressure do not agree with *B* and *C*, they have been omitted in taking the average.
† *B* for 5% was thrown out as defective.

TABLE 8.—4-DAY PERMEABILITY TESTS, 1:3:4 CONCRETE UNDER PRESSURE OF 15 LB. PER SQ. IN. ON BEST AVERAGE SPECIMEN OF EACH SET.

Kind of oil.	Percentage of oil.	Cubic centimeters of water passed through.
No oil.........................	0	38
Richmond Fuel..............	10	62
Road oil No. 6.................	10	87
Liquid Asphalt..	10	185

TABLE 9.—PERMEABILITY TESTS, 1:3 MORTAR.
Average of three specimens. Each pressure on 4 hours.

Kind of oil.	Cubic Centimeters of Water Passed Through.	
	10 lb. per sq. in.	20 lb. per sq. in.
No oil	0	1
Star Fuel	4.0	4.6
Richmond Fuel	19	11.5

SUMMARY OF CONCLUSIONS.

The following conclusions are drawn from the results of these investigations with asphaltic oils:

1.—Oil-mixed concrete containing 5, 10, or 15% of oil, by weight of cement, is more permeable under pressures from 20 to 60 lb. per sq. in., than concrete without the incorporated oil, and oil-mixed mortar containing 10% of oil is more permeable than plain mortar under pressures of 10 and 20 lb. per sq. in.

2.—Oil-mixed mortars containing percentages of oil up to 25 show slightly less absorption of water than plain mortar, except that with the most viscous oils the mortar becomes more absorptive, and absorption is not an index of the permeability. The absorption decreases with the increase in the quantity of oil.

3.—The tensile strength of oil-mixed mortar is decreased considerably below that of plain mortar. The strength decreases with the increase in the quantity of oil.

4.—The compressive strength of oil-mixed mortars follows lines similar to those of the tensile strength, but with a greater decrease in strength with the quantity of oil.

5.—The strength decreases with the viscosity of the oils, and with the most viscous oils it requires considerably more water to keep a mixture at normal consistency than is required in the fluid oil mixtures.

6.—The relative decrease in strength with the increase in the quantity of oil in mortars is less in 50 than in 28 days.

The writers wish to acknowledge their indebtedness to Professor C. B. Wing, M. Am. Soc. C. E., for his aid and advice in the work, and to the Standard Oil Company of San Francisco for its courtesy in furnishing the oil samples.

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PAPERS AND DISCUSSIONS

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EXPERIMENTAL DETERMINATION OF LOSS OF HEAD DUE TO SUDDEN ENLARGEMENT IN CIRCULAR PIPES.*

By W. H. Archer, Esq.

This paper presents a summary of the experimental data and computations therefrom obtained in a series of tests to determine the loss of hydraulic head due to sudden enlargement in cross-section of circular pipes. The question arose in connection with the theory of the jet pump, and its investigation as a thesis led to some interesting conclusions which are herein set forth. Observations were made during December, 1911, and January to March, inclusive, 1912, at the Hydraulic Laboratory, University of California.

Theory.

Theoretically, the determination of loss of head in liquids due to sudden enlargement in cross-section is simple and apparently exact. It is perhaps for this reason that little experimental work has been done to determine the actual losses. For convenience, this theory is hero given, and is as follows:

Let v_1 = velocity, in feet per second, at outlet of smaller pipe;

a_1 = cross-sectional area, in square feet, at outlet of smaller pipe;

p_1 = pressure, in pounds per square inch, at outlet of smaller pipe;

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

γ = density of water, in pounds per cubic foot.

Then $\frac{p}{\gamma}$ = pressure head, in feet of water;

v_2 = velocity, in feet per second, in large pipe, at section where p_2 is a maximum;

p_2 = maximum pressure, in pounds per square inch, in larger pipe;

a_2 = cross-sectional area, in square feet, where p_2 is a maximum;

$l.h.$ = theoretical lost head, in feet of water, due to sudden enlargement;

Q = quantity of water flowing, in cubic feet per second.

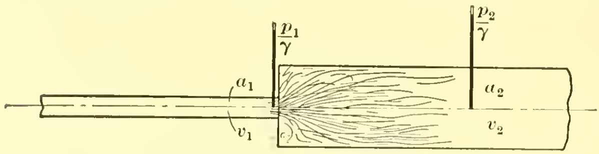


FIG. 1.

The energy relation is:

$$\frac{v_1^2}{2g} + \frac{p_1}{\gamma} = \frac{v_2^2}{2g} + \frac{p_2}{\gamma} + l.h \quad \text{.....................(1)}$$

or

$$\frac{v_1^2}{2g} - \frac{v_2^2}{2g} = l.h. + \frac{p_2}{\gamma} - \frac{p_1}{\gamma} \quad \text{................(2)}$$

or

$$\frac{p_2 - p_1}{\gamma} = \frac{v_1^2 - v_2^2}{2g} - l.h \quad \text{...................(3)}$$

The momentum relation is as follows:

Force = change in momentum, or $(p_2 - p_1)\, a_2 = \frac{Q}{g}\,(v_1 - v_2)$....(4)

By combining Equations (3) and (4) and making continuity substitutions, it is seen that

$$l.h. = \frac{(v_1 - v_2)^2}{2g} \quad \text{.......................(5)}$$

or, in another form.

$$l.h. = \frac{v_1^2}{2g}\left(1 - \frac{a_1}{a_2}\right)^2 \quad \text{..................... (6)}$$

The equation for the value of the actual experimental loss will probably be of the form,

$$L.H. = C\,\frac{v_1^n}{2g}\left(I - \frac{a_1}{a_2}\right)^n \quad \text{...................(7)}$$

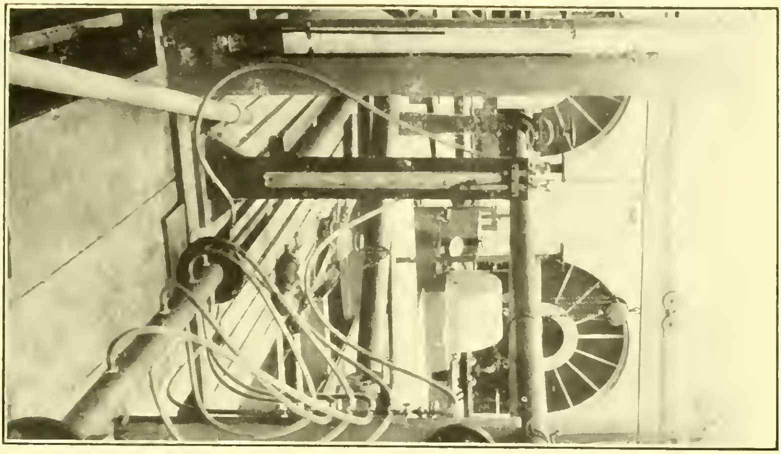


FIG. 1.—VIEW OF APPARATUS FROM OUTLET END.

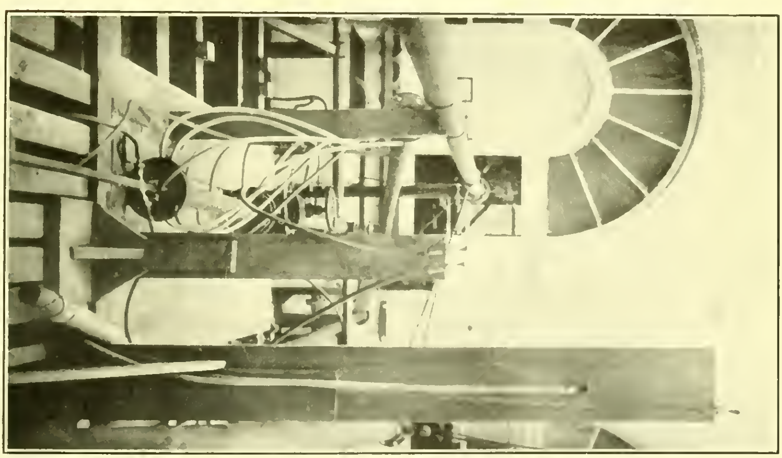


FIG. 2.—VIEW OF APPARATUS FROM INLET END.

C and n being empirical coefficients, to determine the value of which under various conditions was the purpose of the work described.

Equipment Used.

Pipes.—Four sizes of commercial, smooth, brass pipes were used, ranging from 1 to 3 in. in diameter, all being $\frac{1}{16}$ in. thick and 4 ft long. The ends were machined square and even, and burrs were removed with emery paper.

Flanges.—Connections were made with cast-iron flanges, 8 in. in diameter and $\frac{5}{8}$ in. thick, machined to make a water-tight friction fit over the ends of the brass pipe, on which they were driven until just flush. Flanges were bolted together with gaskets between.

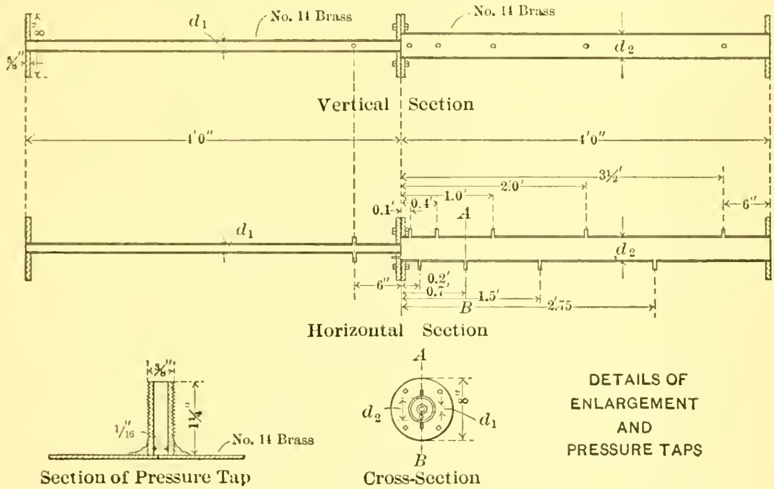


Fig. 2.

Pressure Taps.—Taps were inserted in the sides of the pipe, by which internal pressures were obtained. Taps No. 0 in the small pipe were 6 in. from the plane of enlargement and on opposite sides of the pipe. Taps in the larger pipe were 0.1, 0.2, 0.4, 0.7, 1.0, 1.5, 2.0, 2.75, and 3.5 ft. from the plane of enlargement, and were alternated, that is, No. 2 on one side, No. 3 on the opposite side, etc. Taps were made by drilling $\frac{1}{16}$-in. holes (care being taken to drill into wood so as to leave no burrs inside) and then soldering over these holes pieces of $\frac{1}{8}$-in. brass tube $1\frac{1}{2}$ in. long (Fig. 2). Rubber tubing connected these to the manifold and pressure columns.

Manifold.—The manifold consisted of twelve $\frac{1}{8}$-in. air-cocks screwed into a piece of $\frac{1}{2}$-in. brass tubing 10 in. long. Five cocks were placed on each side for the connections from the pressure taps in the pipe, and one was put on each end for the pressure column connections.

Pressure Columns or **U**-*Tubes.*—Connections were made for using either of two **U**-tubes, depending on the magnitude of pressure difference. One tube, with arms about 3 ft. long, partly filled with mercury, was used for large differences in pressure. The other, having a short arm, $3\frac{1}{2}$ ft., and a long arm, 5 ft. in length, was partly filled with carbon tetra chloride colored red with an aniline dye. Both **U**-tubes were made entirely of $\frac{1}{4}$-in. glass tubing, it being found that the carbon tetra chloride acted on the rubber, causing it to swell until the passage was closed.

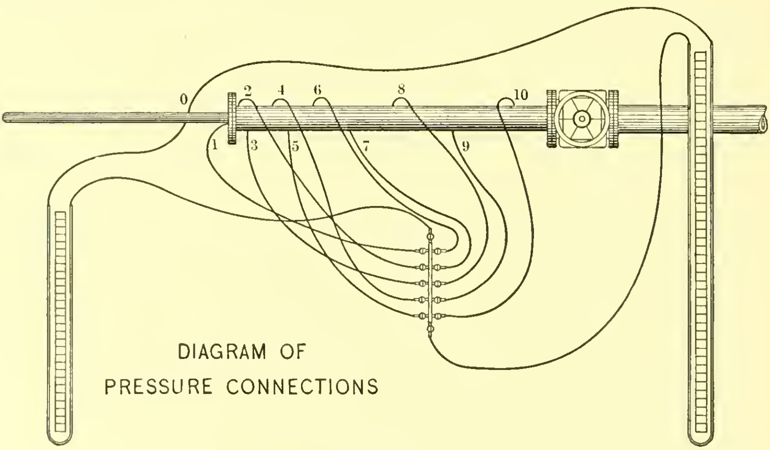


FIG. 3.

In order to obtain a good meniscus on the surface of carbon tetra chloride it is necessary that the glass be absolutely clean. This cleaning was done with a saturated solution of bichromate of potash one part, sulphuric acid three parts and then diluting with six parts of water. It was found necessary to flush all parts of the **U**-tube with running water to remove the cleaning solution thoroughly, otherwise a bothersome scum was formed on the surface of the carbon tetra chloride.

Bleeder cocks were placed at the upper ends of each arm of the **U**-tubes, to allow for the expulsion of air from the tubes and connec-

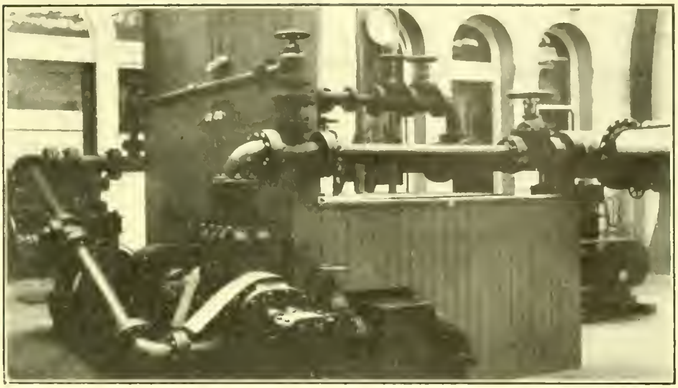


FIG. 1.—QUIMBY SCREW PUMP.
HYDRAULIC LABORATORY, UNIVERSITY OF CALIFORNIA.

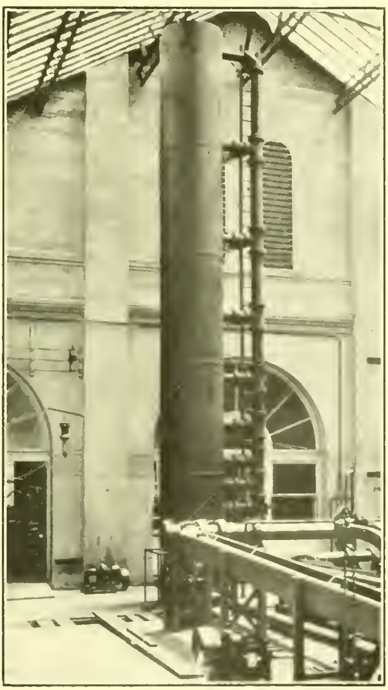


FIG. 2.—STAND-PIPE, HYDRAULIC
LABORATORY, UNIVERSITY
OF CALIFORNIA.

tions. The scales between the arms of the U-tubes were graduated in inches and tenths.

Water Supply.—Water was obtained for the experiments through a horizontal 4-in. pipe about 75 ft. long leading from the laboratory stand-pipe, which is 4 ft. in diameter and 36 ft. high (Plate XLIV, Fig. 2). The height of water in the stand-pipe was maintained with a motor-driven Quimby screw pump (Plate XLIV, Fig. 1), the two cylinders working in series, and drawing water from the main storage reservoir, which is a concrete tank, 18 ft. square and 10 ft. deep, located beneath the floor. Water is kept at varying heights by overflows from the stand-pipe through any required gate in the side thereof, such water returning to the storage reservoir.

Quantity Measurements.—The water was taken from the throttle valve, at the outlet end of the large brass pipe, into a 2½-in. pipe which led to a Y. One arm of the Y discharged into a galvanized-iron tank on platform scales, and the other discharged either into the main reservoir or into a calibrated measuring tank, depending on which way the hinged piece of pipe at the end was swung.

The tank on the scales, with a capacity of about 25 cu. ft., was used for measuring small quantities of water. It was emptied into the main reservoir by a quick-opening valve.

Larger quantities were measured in the calibrated concrete tank, 7 ft. square and 10 ft. deep, using a hook-gauge. A centimeter scale on the hook-gauge was used for a few readings, and an inch scale for the remainder. This tank was also emptied into the main storage reservoir by a 4-in. motor-driven centrifugal pump.

Preliminary Observed Data.—Due to the fact that the iron flanges were driven on the brass pipes, the end diameter was slightly decreased in some cases. The central diameter is taken as the diameter before the flanges were put in place. All measurements were made with micrometer calipers.

No. of pipe.	Average Diameter, in Inches.			Average Areas, in Square Feet.		
	Inlet.	Center.	Outlet.	Inlet.	Center.	Outlet.
1..........	....	0.992	0.982	....	0.00545	0.00526
2..........	1.735	1.745	1.730	0.01641	0.01661	0.01631
3..........	2.488	2.495	2.486	0.03378	0.03395	0.03368
4..........	2.983	2.994	2.993	0.04855	0.04885	0.04885

Letting a_1 represent the outlet area of the small pipe and a_2 the central area of the large pipe, the following values were taken:

No. 1 $a_1 = 0.00526$; $a_2 = 0.00545$
No. 2 $a_1 = 0.01631$; $a_2 = 0.01661$
No. 3 $a_1 = 0.03368$; $a_2 = 0.03395$
No. 4 $a_1 = 0.04885$; $a_2 = 0.04885$

Specific Gravity Determination of Carbon Tetra Chloride.—The specific gravity of the carbon tetra chloride was found by taking the ratio of the length of column of water in one arm of a **U**-tube to the length of column of carbon tetra chloride in the other arm which balanced it.

The average value of four determinations for all the experiments was 1.601, with the exception of the $1\frac{3}{4}$ to $2\frac{1}{2}$-in. combination of pipes, the specific gravity for which (average of three determinations) was 1.570.

These values were used in computing the pressure change, $\frac{p_2 - p_1}{\gamma}$ as follows:

Let S = specific gravity of liquid in **U**-tube;
Let U_1 = scale reading in arm of **U**-tube, where pressure $= p_1$;
Let U_2 = scale reading in arm of **U**-tube connected to tap where pressure $= p_2$.

Then:

If U_1 is greater than U_2, the column of liquid of length $U_1 - U_2$ is balanced by a column of water of length $U_1 - U_2$ and by $\frac{p_2 - p_1}{\gamma}$.

Therefore:

$$\frac{p_2 - p_1}{\gamma} = (S - 1)(U_1 - U_2).$$

In all the determinations, the zero positions, U_0, of the liquid in the columns were carefully noted where $p_1 = p_2$, or when no water was flowing. Then, in most observations, U_1 only was read; and, in nearly all computations, $U_1 - U_2$ was taken as being $2(U_1 - U_0)$. Under the circumstances, this was considered more accurate because there was a perfect meniscus in the arm, U_1, while the surface in the arm, U_2, was broken and irregular. Some readings of U_2 were taken occasionally as a check.

Tables 1 to 5, inclusive, show the values of $\frac{p_2 - p_1}{\gamma}$ computed from readings of U_1 and U_0, and the relation between velocity and $\frac{p_2 - p_1}{\gamma}$ at different points along the length of the large pipe is shown graphically on Figs. 4 to 8, inclusive.

Miscellaneous Data and Constants.—

Specific gravity of mercury.......	13.58
Density of water.................	62.3 lb. per cu. ft.
Measuring tank constants:	
1 cm. gauge height...........	1.523 cu. ft.
1 in. gauge height...........	3.868 " "
Head in stand-pipe above apparatus:	
No. 7, Head	19 ft.
No. 8, "	25 "
No. 9, "	31 "

Method and Order of Taking Observations.

The Quimby screw pump was started, and, when the water was overflowing at the desired gate in the stand-pipe, the maximum range of U_1, the scale reading in the arm of the U-tube connected to the small pipe, was determined for maximum velocity and for some pressure tap in the large pipe where p_2 was near a maximum, say, Tap No. 6. Then, as it was intended to make observations for about ten different velocities for each combination, this range of U_1 was divided into ten steps, and the velocity for each determination was to be regulated by the gate-valve at the outlet end of the large pipe, so that for each successive trial, and for pressure Tap No. 6, the value of U_1 would increase by about one-tenth.

When the velocity had been thus regulated, the initial position of the surface of the water in the measuring tank was observed with a hook-gauge.

The hands of a watch were then set at some even hour, and the instant when the second hand was at 60 the swing pipe was turned so as to divert the water from flowing into the main reservoir and into the measuring tank.

While the water was flowing into the measuring tank at a constant velocity, readings of U_1 and U_2 were taken, when the various cocks in

TABLE 1.

$d_1 = 2.486$ in.; $d_2 = 2.994$ in. Area ratio = 1.451.

Number of Trial.		1	2	3	4	5	6	7	8	9
Velocity, v_1		3.885	7.580	11.060	13.310	5.970	10.000	13.020	14.590	16.580
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various pressure taps......................	No. 1	....	....	....	....	....	....	......	....	....
	No. 2	+0.007	+0.007	+0.005	+0.015	+0.010	+0.012	+0.020	+0.025	+0.051
	No. 3	0.045	0.179	0.394	0.591	0.111	0.326	0.562	0.733	0.915
	No. 4	0.060	0.266	0.577	0.861	0.162	0.477	0.822	1.054	1.370
	No. 5	0.070	0.282	0.631	0.942	0.181	0.522	0.892	1.152	1.490
	No. 6	0.072	0.301	0.648	0.967	0.187	0.535	0.916	1.174	1.520
	No. 7	0.070	0.296	0.633	0.948	0.183	0.520	0.842	1.153	1.486
	No. 8	0.066	0.278	0.590	0.886	0.173	0.490	....	1.075	1.380
	No. 9	0.058	0.248	0.524	0.787	0.153	0.437	....	0.956	1.240
	No. 10	0.051	0.220	0.475	0.726	0.133	0.395	....	0.872	1.136

TABLE 2.

$d_1 = 1.730$ in.; $d_2 = 2.495$ in. Area ratio = 2.077.

Number of Trial.		1	2	3	4	5	6	7	8	9	10
Velocity, v_1		3.820	8.000	10.340	12.530	14.350	16.000	17.380	18.610	20.030	21.050
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	−0.022	−0.044	−0.059	−0.077	−0.084	−0.085	−0.099	−0.100	−0.106	−0.104
	No. 2	−0.021	−0.052	−0.074	−0.101	−0.123	−0.133	−0.151	−0.158	−0.162	−0.162
	No. 3	+0.007	+0.095	+0.168	+0.265	+0.367	+0.476	+0.587	+0.675	+0.787	+0.888
	No. 4	0.034	0.289	0.496	0.745	1.010	1.262	1.532	1.775	2.050	2.280
	No. 5	0.046	0.381	0.638	0.965	1.300	1.622	1.950	2.235	2.590	2.840
	No. 6	0.051	0.415	0.700	1.040	1.395	1.738	2.095	2.375	2.755	3.030
	No. 7	0.051	0.413	0.696	1.032	1.391	1.730	2.080	2.365	2.745	3.020
	No. 8	0.049	0.402	0.680	1.010	1.354	1.690	2.035	2.320	2.690	2.980
	No. 9	0.046	0.365	0.621	0.923	1.235	1.538	1.853	2.120	2.470	2.740
	No. 10	0.043	0.363	0.618	0.923	1.240	1.548	1.860	2.125	2.480	2.740

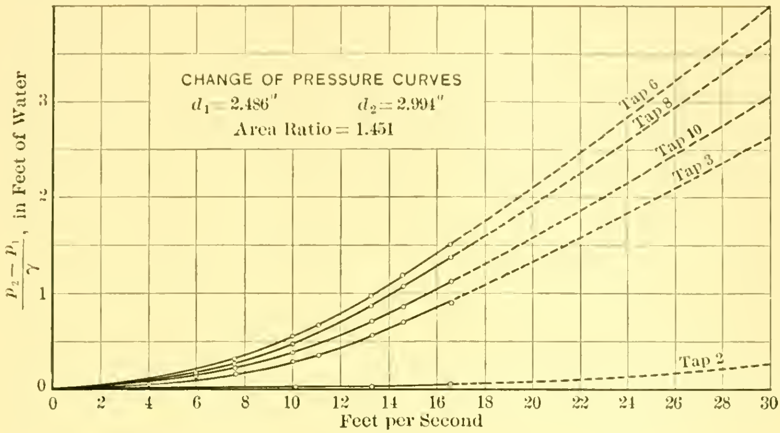

CHANGE OF PRESSURE CURVES
$d_1 = 2.486''$ $d_2 = 2.994''$
Area Ratio = 1.451
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second
Tap 6
Tap 8
Tap 10
Tap 3
Tap 2

FIG. 4.

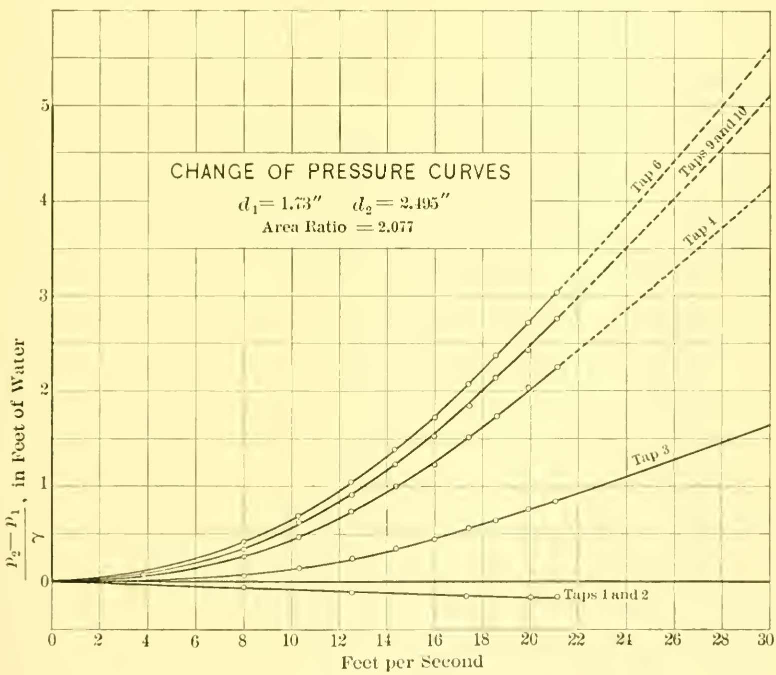

CHANGE OF PRESSURE CURVES
$d_1 = 1.73''$ $d_2 = 2.495''$
Area Ratio = 2.077
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second
Tap 6
Taps 9 and 10
Tap 4
Tap 3
Taps 1 and 2

FIG. 5.

TABLE 3.

$d_1 = 1.730$ in.; $d_2 = 2.994$ in. Area ratio = 2.995.

Number of Trial.		1	2	3	4	5	6	7	8	9	10	11
Velocity, v_1		5.410	7.520	10.250	12.960	15.280	17.120	18.770	20.770	21.120	21.080	23.920
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	—0.027	—0.039	—0.052	—0.060	—0.069	—0.079	—0.081	—0.078	—0.075	—0.075	—0.070
	No. 2	—0.031	—0.044	—0.063	—0.082	—0.092	—0.108	—0.115	—0.123	—0.125	—0.125	—0.130
	No. 3	—0.026	—0.038	—0.045	—0.051	—0.049	—0.056	—0.051	—0.048	—0.030	—0.040	—0.030
	No. 4	+0.045	+0.123	+0.228	+0.415	+0.610	+0.776	+0.960	+1.200	+1.250	+1.230	+1.600
	No. 5	0.128	0.273	0.535	0.880	1.275	1.590	1.960	2.400	2.530	2.480	....
	No. 6	0.154	0.336	0.645	1.060	1.510	1.890	2.300	2.840	2.980	2.920	....
	No. 7	0.163	0.351	0.665	1.090	1.550	1.940	2.410	2.910	3.050	2.990	....
	No. 8	....	0.340	0.655	1.070	1.540	1.920	2.390	2.860	3.010	2.960	....
	No. 9	....	0.337	0.640	1.100	1.500	1.870	2.280	2.810	....	....	....
	No. 10	....	0.325	0.614	1.060	1.450	1.800	2.220	2.720	2.830	2.770	....

TABLE 4.

$d_1 = 0.982$ in.; $d_2 = 2.495$ in. Area ratio = 6.46.

Number of Trial.		1	2	3	4	5	6	7	8	9	10	11
Velocity, v_1		7.020	13.710	19.560	21.380	23.900	24.580	25.120	25.390	25.300	25.820	28.970
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	—0.066	—0.120	—0.153	—0.177	—0.180	—0.175	—0.189	—0.200	—0.134	—0.027	....
	No. 2	—0.071	—0.135	—0.183	—0.234	—0.252	—0.256	—0.269	—0.280	—0.249	—0.289	—0.156
	No. 3	—0.071	—0.134	—0.179	—0.237	—0.238	—0.241	—0.250	—0.256	—0.230	—0.251	—0.270
	No. 4	+0.014	+0.025	+0.110	+0.386	+0.558	+0.602	+0.615	+0.627	+0.630	+0.633	+0.860
	No. 5	0.108	0.343	0.656	1.430	1.875	1.940	2.020	2.065	2.090	2.100	2.760
	No. 6	0.127	0.406	0.748	1.630	2.110	2.200	2.310	2.345	2.370	2.380	....
	No. 7	0.127	0.405	0.755	1.640	2.120	2.210	2.315	2.310	2.375	2.390	....
	No. 8	0.126	0.402	0.753	1.630	2.100	2.200	2.310	2.220	2.370	2.380	....
	No. 9	0.126	0.396	0.740	1.600	2.080	2.170	2.270	2.110	2.335	2.375	....
	No. 10	0.126	0.395	0.736	1.595	2.050	2.155	2.265	2.000	2.310	2.380	....

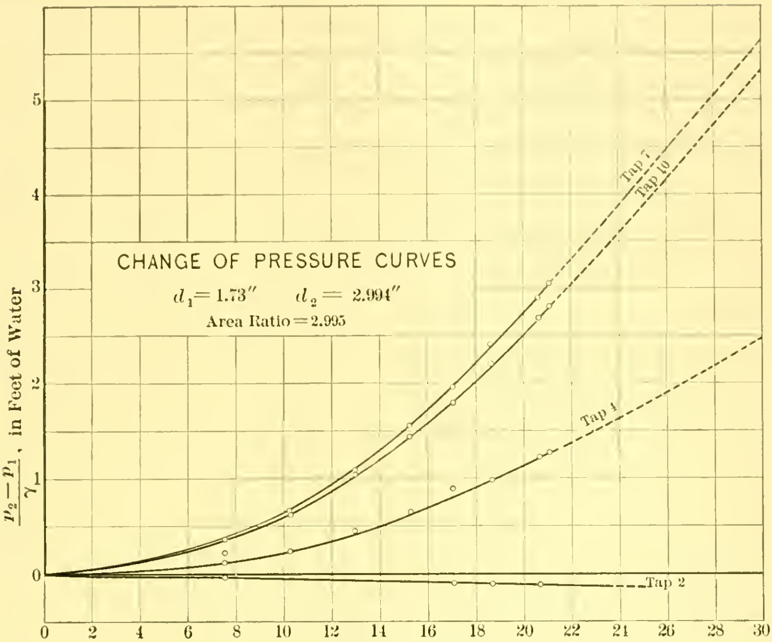

CHANGE OF PRESSURE CURVES
$d_1 = 1.73''$ $d_2 = 2.994''$
Area Ratio = 2.995
Tap 7
Tap 10
Tap 4
Tap 2
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second

Fig. 6.

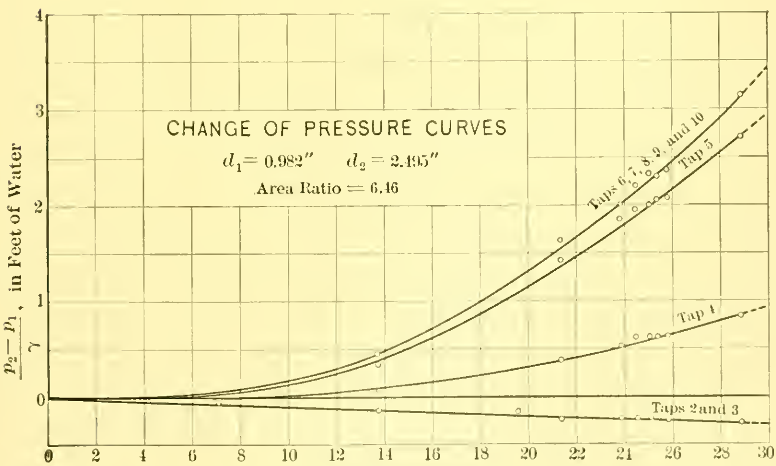

CHANGE OF PRESSURE CURVES
$d_1 = 0.982''$ $d_2 = 2.495''$
Area Ratio = 6.46
Taps 6, 7, 8, 9, and 10
Tap 5
Tap 4
Taps 2 and 3
$\frac{p_2 - p_1}{\gamma}$, in Feet of Water
Feet per Second

Fig. 7.

TABLE 5.

$d_1 = 0.982$ in.; $d_2 = 2.994$ in. Area ratio = 9.22.

Number of Trial.		1	2	3	4	5	6	7	8	9	10	11
Velocity, v_1		6.919	10.370	13.060	14.730	16.410	20.650	25.030	20.540	24.850	27.230	31.580
$\frac{p_2 - p_1}{\gamma}$, or increase in pressure head, in feet of water at various taps.	No. 1	—0.070	—0.119	—0.147	—0.170	—0.173	—0.196	—0.213	—0.193	—0.211	—0.196	—0.193
	No. 2	—0.075	—0.119	—0.161	—0.189	—0.193	—0.228	—0.258	—0.223	—0.251	—0.266	—0.253
	No. 3	—0.075	—0.119	—0.159	—0.183	—0.193	—0.233	—0.253	—0.218	—0.261	—0.261	—0.238
	No. 4	—0.065	—0.101	—0.116	—0.125	—0.132	—0.097	—0.077	—0.107	—0.070	—0.051	+0.018
	No. 5	+0.028	+0.114	+0.215	+0.294	+0.425	+0.725	+1.140	+0.735	+1.130	+1.380	2.005
	No. 6	0.075	0.207	0.360	0.481	0.655	1.092	1.690	1.082	1.680	2.030	2.830
	No. 7	0.081	0.218	0.380	0.520	0.678	1.130	1.740	1.110	1.730	2.110	2.920
	No. 8	0.080	0.219	0.380	0.514	0.680	1.130	1.750	1.115	1.740	2.120	2.920
	No. 9	0.078	0.216	0.376	0.512	0.671	1.126	1.740	1.113	1.728	2.100	2.910
	No. 10	0.076	0.215	0.378	0.512	0.670	1.118	1.672	......	1.710	2.080	2.900

the manifold were turned so as to connect pressure taps No. 1, No. 2, No. 3, etc., successively. Then, at the end of an even number of minutes, the water was diverted back into the storage reservoir and the hook-gauge was set approximately right.

The adjustments of velocity for the next trial were then made while the surface of the water in the measuring tank was coming to rest. Before this was done, however, the controlling gate-valve was entirely closed and the reading U_0 taken. If the liquid in the two columns did not come to the same level, it showed the presence of air in the pressure connections, and doubt was cast on the accuracy of the previous observations. The air was then expelled by opening the bleeder cocks at the tops of the **U**-tube. The velocity was regulated as before, the final tank gauge reading was taken, and the next observation continued as before. Only two or three times during the experiment did the two column arms fail to balance because of the presence of air.

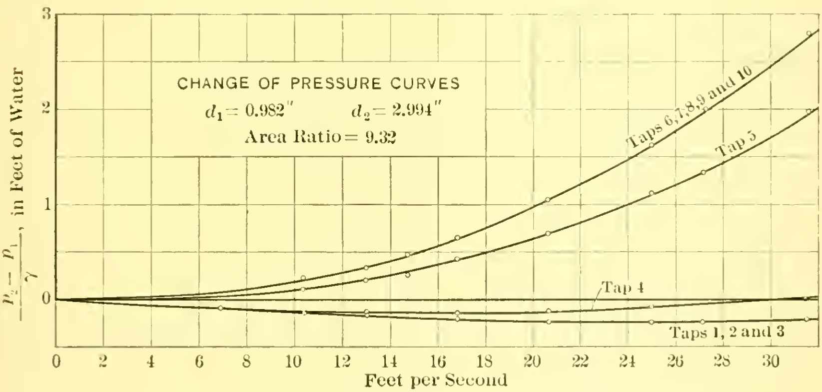


FIG. 8.

The mercury column was used but little, as nearly all the readings were within the range allowed by the long arm of the carbon tetra chloride column, and because the readings of the mercury column were subject to inaccuracy as the rubber tube leading to the **U**-tube came from the top of the manifold so that it would catch any air that might leak into the manifold or tubes connected thereto.

Friction Corrections and Computation of Lost Head.

The experimentally determined value of change in pressure head includes the loss of head due to friction in 6 in. of the smaller pipe and in the length, X, of the larger pipe, where X = the distance from the plane of enlargement to the point of maximum pressure, p_2.

To correct for this, the friction slope for each pipe was computed for several velocities, and lines were plotted on logarithmic paper showing the value of the slope for any velocity in any of the pipes used.

The formula adopted was :*

$$S^{0.54} = \frac{v}{1.32\ C\ r^{0.63}}$$

S = fall, in feet, in friction gradient per foot of length;

v = velocity, in feet per second ;

C = constant, depending on surface of pipe ;

r = hydraulic radius = $\left(\frac{d}{4}\right)$ feet for pipes.

The value of C given by Williams and Hazen for smooth brass pipes is from 1.35 to 1.45; 1.40 was taken.

This formula is based on experimental work† done by A. V. Saph, M. Am. Soc. C. E., and E. W. Schoder, Assoc. M. Am. Soc. C. E. For brass pipes they found:

$H = \frac{0.296}{d^{1.25}} V^{1.75} = 7\%$, where H is the head lost per 1 000 ft. and d is the diameter, in feet. The accuracy of the corrections is indicated by the fact that the experimentally determined hydraulic gradients (Fig. 10) become horizontal when thus corrected.

Computation of Lost Head.—Using the following notation:

v_1 = velocity of water in small pipe;

v_2 = velocity of water in large pipe;

a_1 and a_2 = respective cross-section areas, in square feet;

g = acceleration due to gravity = 32.2 ft. per second per second;

h = total friction head correction;

$\frac{p_2 - p_1}{\gamma}$ = measured change in pressure head ;

$L.H.$ = actual experimental lost head due to sudden enlargement.

$$L.H. = \frac{v_1^2 - v_2^2}{2g} - \frac{p_2 - p_1}{\gamma} - h\ ;$$

but

$$\frac{v_1^2 - v_2^2}{2g} = \frac{v_1^2}{2g}\left(1 - \frac{(a_1)^2}{(a_2)}\right)$$

Therefore

$$L.H. = \frac{v_1^2}{2g}\left(1 - \frac{(a_1)^2}{(a_2)}\right) - \frac{(p_2 - p_1)}{\gamma} - h.$$

* Taken from Williams and Hazen's "Hydraulic Tables."

† *Transactions*, Am. Soc. C. E., Vol. LI, p. 253.

Using this formula, the results are shown in Tables 6 to 10, inclusive.

TABLE 6.

$d_1 = 2.486$ in.; $d_2 = 2.944$ in. Area ratio = 1.451.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.......	3.885	2.680	0.072	0.038
2.......	7.580	5.220	0.301	0.098
3.......	11.060	7.620	0.648	0.212
4.......	13.310	9.180	0.967	0.280
5.......	5.970	4.120	0.187	0.060
6.......	10.000	6.900	0.535	0.163
7.......	13.020	8.980	0.916	0.286
8.......	14.590	10.060	1.174	0.330
9.......	16.580	11.440	1.520	0.429

TABLE 7.

$d_1 = 1.730$ in.; $d_2 = 2.495$ in. Area ratio = 2.077.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.......	3.82	1.837	0.0513	0.108
2.......	8.00	3.842	0.4150	0.259
3.......	10.34	4.970	0.700	0.434
4.......	12.53	6.020	1.04	0.624
5.......	14.35	6.900	1.395	0.787
6.......	16.00	7.690	1.738	0.977
7.......	17.38	8.36	2.095	1.120
8.......	18.61	8.94	2.375	1.319
9.......	20.03	9.63	2.755	1.515
10.......	21.05	10.12	3.03	1.694

TABLE 8.

$d_1 = 1.730$ in.; $d_2 = 2.944$ in. Area ratio = 2.995.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1......	5.41	1.088	0.163	0.208
2......	7.52	2.510	0.351	0.385
3......	10.25	3.424	0.660	0.660
4......	12.96	4.33	1.090	1.033
5......	15.28	5.105	1.55	1.411
6......	17.12	5.72	1.94	1.775
7......	18.77	6.27	2.41	1.982
8......	.....	......	......	......
9......	21.12	7.055	3.05	2.635
10......	.....	......	......	......
11......	23.92	7.990	4.19	3.100

TABLE 9.

$d_1 = 0.982$ in.; $d_2 = 2.495$ in. Area ratio = 6.46.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1.......	7.02	1.086	0.127	0.510
2.......	......	......	......	......
3.......	......	......	......	.....
4.......	21.38	3.31	1.640	4.420
5.......	23.90	3.70	2.120	5.461
6.......	24.58	3.805	2.210	5.816
7.......	25.12	3.890	2.310	6.072
8.......	......	......	......	.... .
9.......	......	......	......	......
10.......	25.82	4.00	2.39	6.488
11......	28.97	4.48	3.20	8.006

TABLE 10.

$d_1 = 0.982$ in.; $d_2 = 2.994$ in. Area ratio = 9.32.

Trial.	v_1 (from data).	v_2 (from data).	$\frac{p_2 - p_1}{\gamma}$ (from data).	L.H. (computed).
1......	6.919	0.744	0.0803	0.555
2......	10.37	1.17	0.219	1.209
3......	13.06	1.40	0.380	1.894
4......	14.73	1.58	0.514	2.384
5.....	16.41	1.77	0.680	2.923
6......	Near duplicate of other trial.			
7......	25.03	2.69	1.75	6.721
8......	Near duplicate of other trials.			
9......	" "	" "	"	
10......	27.23	2.93	2.12	7.854
11......	31.58	3.40	2.92	10.646

Determination of Formula for Lost Head.

These values of v_1 and lost head, Tables 6 to 10, inclusive, were plotted three times, independently, on logarithmic cross-section paper. The plotted points determined lines which were not only straight but nearly parallel, which indicated that the lost head varied as a constant power of the velocity. The value of this exponent (n) which, of course, was the tangent of the angle of inclination of the parallel lines with the velocity axis, was 1.919, and was determined by taking an average of the fifteen values obtained by the three trials of plotting of five lines each. In short, *L.H.* varied as $v_1^{1.919}$.

Also: because, by theory, $l.h. = \frac{v_1^2}{2g}\left(1 - \frac{a_1}{a_2}\right)^2$, assume that

$$L.H. = c\, v_1^n \left(1 - \frac{a_1}{a_2}\right)^n.$$

This relation was proven to be true by plotting on logarithmic cross-section paper the values of lost head as ordinates against the corresponding values of $\left(1 - \frac{a_1}{a_2}\right)$ for a constant velocity. All the points lay very close to a straight line with slope = 1.919.

The production of this line to the axis, where $\left(1 - \frac{a_1}{a_2}\right)$ was equal to unity, and thence to such a value that v_1 was unity, gave the value of the constant, c.

From such construction, $C = 0.01705$. Hence the relationship appears to be

$$L.H. = 0.01705\, v_1^{1.919} \left(1 - \frac{a_1}{a_2}\right)^{1.919}$$

or

$$L.H. = 1.098\, \frac{v_1^{1.919}}{2g} \left(1 - \frac{a_1}{a_2}\right)^{1.919}$$

Relation of Theoretical to Actual Loss of Head.

$$L.H. = \text{Actual loss} = K\, V_1^{1.919}$$

$$l.h. = \text{Theoretical loss} = k\, v_1^2$$

$$\frac{L.H.}{l.h.} = \frac{K\, V_1^{1.919}}{k\, v_1^2} = K'\, v_1^{-0.081}$$

or, if $B = K'\, v_1^{-0.081}$ where B is a variable coefficient.

$$\text{then } L.H. = B \times (l.h.) = B\, \frac{(v_1 - v_2)^2}{2g}.$$

The value of B, or the ratio of actual loss to theoretical loss at any velocity, v_1, may be shown on logarithmic cross-section paper, for any desired ratio of pipe areas, by drawing lines with slope = — 0.081 at such positions that their intersections with lines of actual lost head for the same area ratios occur at velocities of such values that the theoretical loss for corresponding area ratios is unity. In other words, determine the value of v_1 to make the theoretical loss unity for any desired area ratio, and then the value of the actual loss at that velocity is a point on the line with slope = — 0.081, showing the value of B for that area ratio and for any velocity. Table 11 shows the values of B for different area ratios and velocities.

TABLE 11.—VALUE OF COEFFICIENT, B.

$$\text{True lost head} = B\left(\frac{v_1 - v_2}{2g}\right)^2.$$

Vel. v_1 / $a_1 \div a_2$	2	4	6	8	10	12	15	20	25	30	40	50	60	70	80
1 : 1¼	1.225	1.162	1.123	1.092	1.075	1.060	1.038	1.017	0.994	0.981	0.959	0.941	0.926	0.915	0.903
1 : 1½	1.155	1.087	1.057	1.025	1.001	0.991	0.976	0.953	0.938	0.923	0.901	0.886	0.871	0.860	0.850
1 : 2	1.090	1.030	0.998	0.975	0.958	0.943	0.928	0.905	0.889	0.876	0.855	0.839	0.826	0.816	0.807
1 : 2½	1.068	1.012	0.977	0.954	0.938	0.924	0.906	0.886	0.870	0.857	0.837	0.821	0.809	0.798	0.789
1 : 3	1.063	1.002	0.970	0.947	0.930	0.917	0.900	0.880	0.863	0.849	0.830	0.816	0.802	0.791	0.783
1 : 4	1.055	0.996	9.965	0.943	0.925	0.911	0.893	0.876	0.858	0.846	0.827	0.811	0.799	0.788	0.780
1 : 5	1.049	0.911	0.960	0.938	0.920	0.906	0.890	0.870	0.852	0.840	0.820	0.805	0.792	0.782	0.774
1 : 10	1.040	0.983	0.952	0.930	0.913	0.899	0.883	0.869	0.847	0.835	0.815	0.800	0.788	0.779	0.769
1 : 20	1.033	0.979	0.947	0.925	0.907	0.893	0.878	0.858	0.841	0.829	0.810	0.794	0.782	0.772	0.764
1 : ∞	1.022	0.968	0.937	0.915	0.898	0.884	0.869	0.848	0.832	0.820	0.800	0.785	0.773	0.763	0.754

Hydraulic Gradients.

Tables 12 to 16, inclusive, show the data from change of pressure curves, Figs. 4 to 8, inclusive, for hydraulic gradients, not corrected for friction, the values of $\frac{p_2 - p_1}{\gamma}$ being in feet of water. Fig. 9 shows a typical set of gradients plotted from Table 12.

TABLE 12.—Hydraulic Gradients.

Combination No. 1 : $\frac{a_2}{a_1} = 1.451.$

v_1	Tap Number.									
	1	2	3	4	5	6	7	8	9	10
5	.....	+0.003	+0.80	+0.13	+0.15	+0.15	....	+0.12	....	+0.11
10	.....	+0.011	+0.30	+0.43	+0.51	+0.54	....	+0.44	....	+0.39
15	.....	+0.035	+0.77	+1.11	+1.22	+1.28	....	+1.12	....	+0.93
20 (from curves produced)	.....	+0.100	+1.33	+1.90	+2.10	+2.10	....	+2.00	....	+1.60
25 (from curves produced)	.....	+0.176	+1.94	+2.75	+3.04	+3.00	....	+2.96	....	+2.30

TABLE 13.—Hydraulic Gradients.

Combination No. 2 : $\frac{a_2}{a_1} = 2.077.$

v_1	Tap Number.									
	1	2	3	4	5	6	7	8	9	10
5	—0.03	—0.028	+0.02	+0.10	+0.11	+0.16	.....	.....	+0 12	....
10	—0.06	—0.075	+0.13	+0.43	+0.61	+0.65	.....	.....	+0.58	....
15	—0.087	—0.128	+0.40	+1.13	+1.43	+1.53	.....	....	+1.36	....
20	—0.105	—0.1625	+0.79	+2.05	+2.59	+2.75	.....	.....	+2.47	....
30 (from curves produced)	....	....	+1.75	+4.20	+5.25	+5.60	.....	.....	+5.10	....

TABLE 14.—Hydraulic Gradients.

Combination No. 3 : $\frac{a_2}{a_1} = 2.995.$

v_1	Tap Number.									
	1	2	3	4	5	6	7	8		10
5	—0.025	—0.025	—0.025	+0.04	+0.12	+0.20	+0.21	....	.. .	+0.21
10	—0.05	—0.063	—0.042	+0.22	+0.50	+0.60	+0.63	....	....	+0.55
15	—0.07	—0.095	—0.053	+0.63	+1.24	+1.50	+1.50	....	....	+1.43
20	—0.08	—0.120	—0.051	+1.14	+2.26	+2.66	+2.75	....	....	+2.57
25	—0.07	0.130	—0.025	+1.75	+3.36	+4.02	+4.15	....	....	+3.83
30 from curves produced)	—0.06	—0.140	0.	+2.42	+4.53	+5.32	+5.62	....	....	+5.21

TABLE 15.—HYDRAULIC GRADIENTS.

Combination No. 4 : $\frac{a_2}{a_1} = 6.46$.

v_1	Tap Number.						
	1	2	3	4	5	6 and 7	9 and 10
5..........	—0.05	—0.05	—0.05	+0.01	+0.08	+0.10	+0.10
10..........	—0.09	—0.103	—0.10	+0.02	+0.20	+0.25	+0.25
15..........	—0.12	—0.157	—0.15	+0.10	+0.52	+0.62	+0.58
20..........	—0.17	—0.21	—0.20	+0.30	+1.23	+1.40	+1.35
25..........	—0.18	—0.26	—0.24	+0.60	+2.03	+2.30	+2.25
30..........	......	—0.10	—0.26	+0.94	+2.89	+3.26	+3.20

TABLE 16.—HYDRAULIC GRADIENTS.

Combination No. 5 : $\frac{a_2}{a_1} = 9.32$.

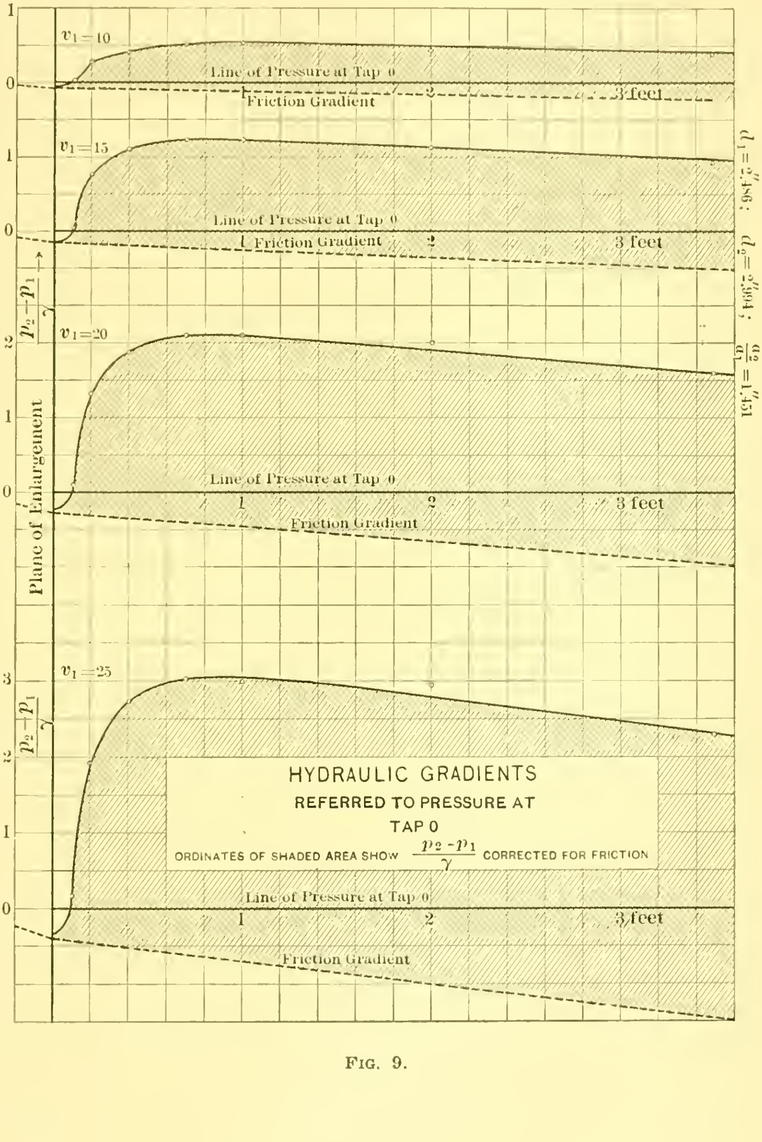
v_1	Tap Number.								
	1	2	3	4	5	6	7	8	9
5.........	—0.05	—0.55	—0.053	—0.045	+0.02	+0.04	+0.07	+0.07	+0.07
10.........	—0.106	—0.12	—0.12	—0.09	+0.11	+0.18	+0.20	+0.20	+0.20
15.........	—0.155	—0.173	—0.175	—0.125	+0.32	+0.50	+0.53	+0.53	+0.51
20.........	—0.197	—0.222	—0.225	—0.121	+0.70	+1.03	+1.11	+1.11	+1.10
25.........	—0.212	—0.258	—0.258	—0.076	+1.16	+1.72	+1.77	+1.77	+1.75
30.........	—0.199	—0.260	—0.250	—0.012	+1.75	+2.46	+2.48	+2.48	+2.47

Fig. 10 shows a set of gradients for a common velocity and different area ratios. The friction effect has been corrected by increasing the ordinates by an amount equal to the computed friction drop from Tap 0, or the reference pressure tap. It will be noted that the maximum change in pressure occurs when the area ratio is about 1 to 2 or 1 to 3, which is what would be expected from the theory as deduced by the application of the laws of maxima and minima.

DISTANCE OF MAXIMUM p_2 FROM PLANE OF ENLARGEMENT.

It was readily seen that this distance (call it x) would undoubtedly depend on the thickness ($r_2 - r_1$) of the water between the stream from the smaller pipe, and the inside surface of the large pipe, or would vary as some function of ($d_2 - d_1$).

For each of the combinations tested the value of x was determined within limits from the gradients and from inspection of the data. These limits were plotted on logarithmic paper with ($d_2 - d_1$) in



$v_1=10$
Line of Pressure at Tap 0
Friction Gradient
3 feet
$v_1=15$
Line of Pressure at Tap 0
1 Friction Gradient
2
3 feet
$v_1=20$
Line of Pressure at Tap 0
1
2
3 feet
Friction Gradient
$v_1=25$
Line of Pressure at Tap 0
1
2
3 feet
Friction Gradient
$\frac{p_2-p_1}{\gamma}$
Plane of Enlargement
$d_1 = 2''.486$; $d_2 = 2''.994$; $\frac{d_2}{d_1} = 1''.451$
HYDRAULIC GRADIENTS
REFERRED TO PRESSURE AT
TAP 0
ORDINATES OF SHADED AREA SHOW $\frac{p_2-p_1}{\gamma}$ CORRECTED FOR FRICTION

FIG. 9.

inches as abscissas. It was found that a straight line with slope = 0.4 would pass very nearly through the average values, which would indicate that $x = K\,(d_2 - d_1)^{0.4}$.

From the line on logarithmic paper it is seen that when $(d_2 - d_1) = 1$, then $x = 1.45 = K$.

$$\text{Hence } x = 1.45\,(d_2 - d_1)^{0.4}$$

where d_2 and d_1 are diameters, in inches.

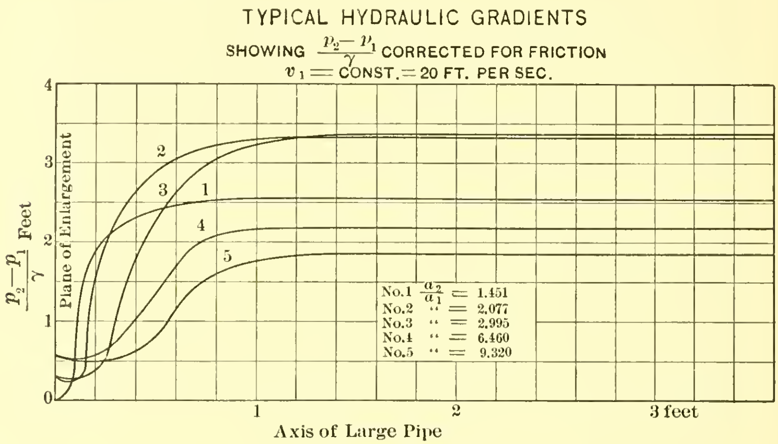


FIG. 10.

CONCLUSIONS.

The formulas thus obtained are based on data within the following limits:

v_1 from 3.82 to 31.58 ft. per sec.

v_2 from 0.07 to 11.44 ft. per sec.

$\frac{a_2}{a_1}$ from 1.451 to 9.32.

It is also worthy of note that few reliable results were obtained with values of v_1 less than 6 ft. per sec.

From the values of B it is seen that, within all ordinary limits of values of velocity and area ratios, the actual loss is less than the theoretical loss. This, at first, seems peculiar, but it is well known that velocity head may be changed to pressure head, with more or less attendant loss, by using a diverging passage to decrease the velocity

gradually and cause as little disturbance of stream lines as possible. In practice, this is accomplished by diffusers made of solid material. In the present instance, such action occurs with the dead water at the sides of the stream entering with velocity, v_1, having such diffusing effect. This explanation is borne out by the fact that the value of B decreases, or the amount by which the theoretical loss is lessened increases, as the velocity increases, or as the relative "deadness" of the side water increases. B also decreases as the area ratio increases, or as the quantity of dead water increases.

Mr. A. H. Gibson* deduces the same theoretical loss as given herein, and substantiates it with some experimental work he had done. Such results seem to agree closely with the theory, but, unfortunately, the maximum, p_2, was taken too close to the plane of enlargement, resulting in a smaller value of $\frac{p_2 - p_1}{\gamma}$ and hence a larger lost head. He used the value of $x = 10$ in., which, according to the formula, $x = 1.45\ (d_2 - d_1)^{0.4}$ should have been 1.7 ft. or 20.4 in. for maximum p_2, as $(d_2 - d_1) = 1.5$ in. The statement is made that by taking as p_2 the value obtained at a point 4 in. from the enlargement, the loss is about 20% greater than the theoretical, which agrees closely with the results herewith presented, as the actual loss decreases as p_2 increases, other things being equal. Had p_2 been taken at its maximum, losses from 0 to 20% less than theoretical, depending on the velocity, would doubtless have been observed. It is probable that the values of p_2 were taken at about the point where the loss changes from greater than theoretical to less than theoretical.

Summary of Results.—The summary of results is as follows:

(1) Lost head due to sudden enlargement $= L.H.$

$$L.H. = 1.098\ \frac{v_1^{1.919}}{2g}\left(1 - \frac{a_1}{a_2}\right)^{1.919}$$

$$\text{or } L.H. = 1.098\ \frac{(v_1 - v_2)^{1.919}}{2g}$$

$$\text{or } L.H. = \frac{B\ (v_1 - v_2)^2}{2g},$$ where B is a variable coefficient with values given in Table 11.

* "Hydraulics and Its Applications," published in 1908.

(2) Distance of maximum p_2 from plane of enlargement $= x$

$$x = 1.45\ (d_2 - d_1)^{0.4}$$

where d_2 and d_1 are diameters in inches.

The writer takes this opportunity to thank all who helped bring this work to completion. The experimental work was done under the general supervision of Professor J. N. Le Conte, who gave many helpful suggestions in the computations as well. H. T. Cory, M. Am. Soc. C. E., also gave much valuable advice in connection with the coordination of the results and their preparation for presentation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

COLORADO RIVER SIPHON.

By George Schobinger, Jun. Am. Soc. C. E.

To be Presented May 7th, 1913.

Yuma is on the Arizona side of the Colorado River, a short distance below the mouth of the Gila River and about 25 miles north of the Mexican boundary. The bottom lands on both sides of these two rivers, leveed for protection against overflow, together with some mesa land, form the irrigable land of the Yuma Project of the United States Reclamation Service. The water for irrigation is taken from the Colorado at Laguna Dam, a diversion weir 14 miles above Yuma. To conduct the water to the Yuma Valley and Mesa, which is that portion of the Project on the Arizona side below Yuma, it was necessary to construct main canals to, and provide means of crossing, either the Colorado or the Gila. Both rivers have unstable beds, and are subject to sudden floods, which scour out deep channels. Owing to the relative elevations of the water in the canals and rivers, and the great variation between flood stage and low-water stage, a crossing by an aqueduct over either river would have been impossible, and an inverted siphon crossing was the only practicable one. For reasons of economy in the construction of both canal and crossing, the Colorado River siphon was selected. As may be seen on the general map, Fig. 1, the siphon is near Yuma, and a short distance below the Southern Pacific Railroad Bridge. The river at this point flows be-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings,* and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

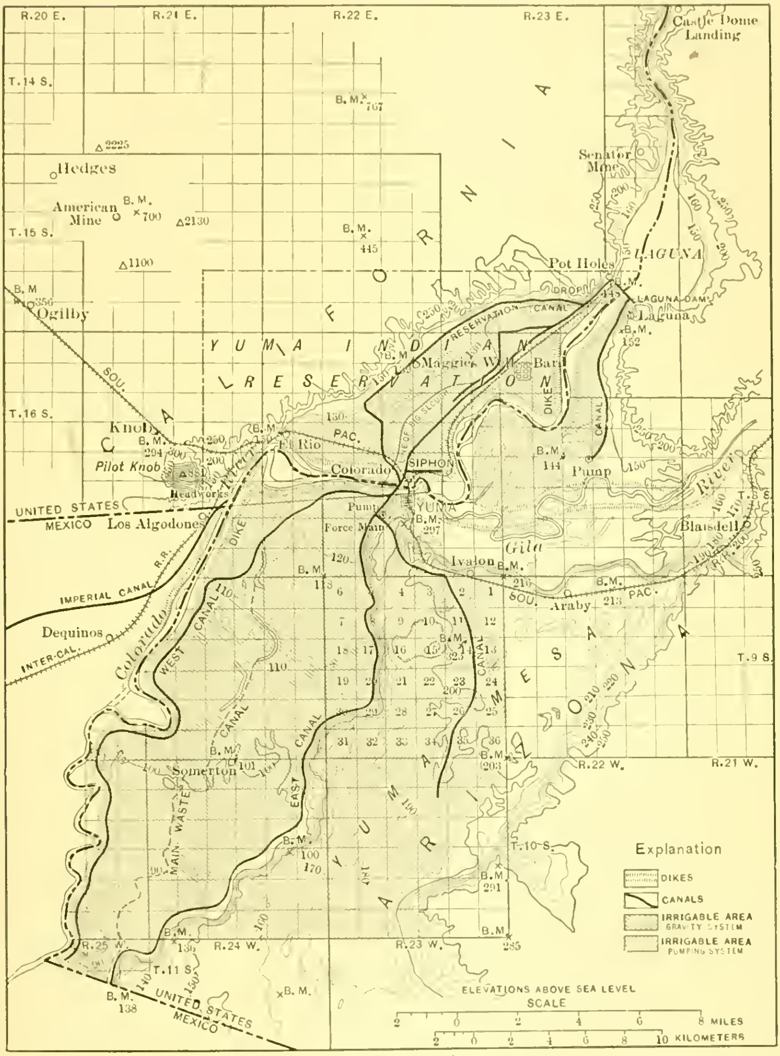
tween high banks which restrict its meandering tendencies and permit a shorter tunnel than would be possible at other points.

The variation between extreme flood and low-water surface elevations has been as great as 18 ft. This, however, does not represent the variation in cross-section; it has been said that for every foot the river rises it scours out its bed 2 ft., and the cross-sections on Fig. 7 illustrate the approximate truth of the statement. The deepest scour within recent years is shown; this section was taken at the river gauging station 600 ft. below the line of the tunnel, the width of the river being the same at both points. At the periodic high-flood years the scour is always approximately the same. In the intermediate low-flood years the 30-ft. silt blanket dropped by the falling river of the preceding years is only partly removed.

Borings taken on the line of the tunnel indicated a bed of soft sandstone underlying the silt. It was thought that, by working at a sufficient depth, open tunneling could be carried on without great interference from seepage water. Therefore, preparations were made for sinking shafts on each side of the river with the intention of following this method of construction.

Before describing the construction in detail, a few words may be said of the general dimensions of the work. The California shaft is 130 ft. from the center of the levee, and 250 ft. from the Southern Pacific Railroad. The Arizona shaft is in high ground, 200 ft. from the river, and 955 ft. from the California shaft. The canals conducting the water to and from the shafts have a bottom width of 80 ft., a water depth of 7 ft., and carrying 1 400 sec-ft., at full capacity, which is sufficient for the irrigation of about 100 000 acres. The siphon was designed to operate at full capacity with a loss of head of 2 ft. When operating at less than full capacity the same loss of head is obtained by throttling the flow at the intake; thus the water surface at the intake and outlet are maintained uniformly at Elevations 132 and 130, respectively. The inside diameter of the California shaft is 17 ft., and that of the Arizona shaft is 23 ft. The elevation of the tunnel flow line is + 49 at the shafts and + 47 in the middle of the river.

The assumed maximum conditions of loading to which the tunnel is subjected are, (*a*) tunnel empty, with pressure on the outside only, due to a hydrostatic head of 74 ft., at high stage of the river, and (*b*)



GENERAL MAP
YUMA PROJECT ARIZONA-CALIFORNIA
UNITED STATES RECLAMATION SERVICE

FIG. 1.

system in operation, at low stage of the river, with pressure on the inside due to a hydrostatic head of 76 ft., partly balanced by a head on the outside of 59 ft. Under these conditions, both the tensile and compressive stresses in the 2-ft. thick tunnel lining are very low. The many uncertainties as to the actual loading from the material through which the tunnel passes, and the desirability of a fairly water-tight job, fully justify the thickness of the tunnel walls as constructed.

The stresses in the completed shafts due to the static loads would be considerably less than those in the tunnel, as the lining of both shafts is 3½ ft. thick and is further reinforced at the bottom by the quarter-turn lining. The most severe conditions to which the shafts were subjected were those due to the method of construction, which was to sink them as open caissons. As these would be largely a matter of conjecture, there would be small profit in attempting to compute the stresses. Important determining factors in fixing the thickness of the lining were the probable skin friction and the weight required to sink the caisson. As a guide in determining the thickness necessary to resist the stresses during the sinking and to furnish weight to overcome the skin friction and give a water-tight shaft, comparison was made with a number of like structures built under similar conditions. These showed a variation in thickness of walls of from 2½ to 6 ft., and, in the skin friction overcome in sinking them, of from 300 to 700 lb. per sq. ft. The thickness was fixed at 3½ ft., with no reinforcement, except a small number of short rods used to bond adjoining sections.

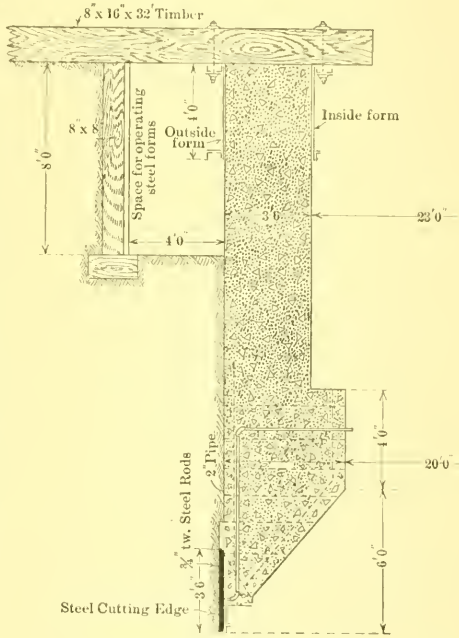
Arizona Shaft.

The cutting edge consisted of a cylindrical skin plate ½ in. thick and 3½ ft. wide, bent on a circumference of 30 ft. diameter. It is reinforced near the bottom edge by a 10-in., 25-lb. channel with its web riveted to the inside of the plate. To the top flange of the channel is riveted a horizontal 14½ by ½-in. shoe plate, stiffened at its inner edge by a 3 by 3 by $\frac{7}{16}$-in. angle, and connected to the channel at sixteen points on the circumference by vertical plate and angle brackets. The outside of the shaft concrete is flush with the outside of the circumferential plate. On the inside, the concrete batters in from the shoe plate to a diameter of 20 ft. at a point 6 ft. above the cutting edge, giving a thickness of concrete of 5 ft. at this point. The inside face is vertical for 4 ft. above this, and then steps out to the normal

inside diameter of the shaft, which is 23 ft. The rivet heads on the outside of the cutting edge skin plate were countersunk, in order to reduce the friction.

In preparing to sink the shaft, a pit, some 40 ft. in diameter and 12 ft. deep, was excavated, using teams and scrapers. The bottom was leveled and the cutting edge assembled and placed. Wooden forms were constructed for the lower 10 ft. of the shaft, and the concrete was placed. The steel forms, which were used throughout the shaft construction, were then set in place.

To support the forms, twelve 8 by 16-in. timbers, 32 ft. long, were set on radial lines projecting as cantilevers over the shaft walls, the outer ends being firmly anchored in the ground. Bolts passing through these timbers carried the forms. As may be seen on the section, Fig. 2, the inside of the shaft was thus left open, so that excavation could be carried on without interference by other work. A passage was also left on the outside of the shaft to give access to the outside form.



HALF SECTIONAL ELEVATION

FIG. 2.

The forms were built in segments of a cylindrical surface, with $\frac{3}{16}$-in. plate, 4 ft. high, stiffened at top and bottom by circular ribs of built-up channels with horizontal web. The segments were connected so that, when they were assembled and in place, the circumference of the whole could be increased or decreased by turnbuckles. Thus, when the concrete had been built up, as previously described, the inside and outside forms were fitted to the completed concrete, with a vertical lap of about 2 in., and were plumbed, and clamped in place

by tightening the turnbuckles. No braces, ties, spreaders, or other means of support were required. When the forms had been filled with concrete, the hanger-bolts were released; excavation proceeded through the open space in the center of the shaft, and, as the caisson sank, the forms traveled with it. In preparing for a new lift of concrete, the turnbuckles were released, the forms were pried from the face of the concrete, raised to their original level with the derrick or tackles supported from the cantilever timbers, and reset as before. In this way a shaft wall of uniform thickness and very smooth surface was obtained.

A ½-cu. yd. Smith mixer, operated by steam, was used. Sand and stone were fed to the mixer from a two-pocket bin, having a capacity of 20 cu. yd., and were discharged into the measuring hopper. The bin was replenished from the stock piles directly back of it, by a derrick with a grab-bucket. This could be done at times when the derrick was not needed for other work. The concrete was mixed in the proportions 1 : 2½ : 5, Iola and Riverside Portland cement being used. The concrete was chuted from the mixer directly into the forms, several movable chutes of different lengths being used.

The excavation was carried on by hand, with pick and shovel, the spoil being removed by the derrick with a 1-cu. yd. bucket. The core was removed, the cutting edge undermined, and the shaft allowed to sink, the concrete being built up at the surface in 3½-ft. lifts as the work progressed.

At Elevation 120 ground-water was encountered, at first in small quantities, but rapidly increasing with the depth below this plane. Pumps were put in place to keep the pit dry, and hand excavation was continued to a further depth of about 50 ft. Here the excess hydrostatic head on the outside of the shaft was sufficient to cause sudden inrushes of water and sand under the cutting edge, and it was found necessary to change the method of excavation. The core was now removed by clam-shells to a depth of from 5 to 10 ft. below the cutting edge. Divers then drilled holes in the supporting ring under the cutting edge and placed small charges of dynamite. These, when fired, caused the shaft to sink varying distances.

When the cutting edge had penetrated some distance into the bed of soft sandstone indicated by the borings, attempts were made to seal the shaft to the undisturbed hard material by sinking pipes from

the surface outside of the caisson walls and forcing grout through them into the running sand. It was thought that by impregnating with grout the soft ground surrounding and for some distance above the tunnel opening, the flow of water might be sufficiently cut off to permit of open tunneling into unfissured, less pervious ground. After the grouting was complete a concrete plug was placed in the bottom of the shaft under water, and the water was pumped out. Preparations were made for tunneling operations. Cage guides and a gallows frame were constructed and hoisting machinery was erected. An open crib was built on top of the shaft, and a working floor laid. Tracks were laid from the mixer to the shaft and from the shaft to the spoil bank. While work was under way cutting through the shaft walls for a tunnel opening, other "blows" of water and sand occurred, and at the last of these the work was flooded. Further efforts at open tunneling were then abandoned, it being evident that the grout had not efficiently consolidated the material surrounding the shaft, and that the soft and porous nature of the ground and the high head of water would make it necessary to work under compressed air.

The foregoing method of consolidating running sand in foundation and similar work has been described as successful in several engineering writings. The outcome in this case showed conditions similar to those found in the construction of the New York tunnels of the Pennsylvania Railroad. The following is quoted from a discussion by the late C. L. Harrison, M. Am. Soc. C. E.*

"Grout was used extensively in the face in an effort to consolidate the material. In digging out the sand into which the grout had been injected under pressure, it was found to be collected in masses and not generally distributed; this was true whether the sand contained a large or a small percentage of voids. The interstices between the grains of sand were too small to permit the free flow of grout into them; however, grouting in the face did compact the material to some extent and reduce the flow of air through it."

Francis L. Sellew, M. Am. Soc. C. E., says:†

"In the writer's judgment, it is impracticable to so consolidate fine sand by the injection of grout that the resulting material will serve any useful purpose. While sufficient pressure may be used to cause penetration, the fluid will move at such low velocities that the grout will set up before any appreciable distance has been traveled.

* *Transactions*, Am. Soc. C. E., Vol. LXIX, p. 400.

† "The Colorado River Siphon," *Engineering News*, August 29th, 1912.

It is believed that success attained by this method will be confined to coarse materials."

California Shaft.

The work at the California shaft will not be described in detail, as the methods and results were similar to those at the Arizona shaft, except as to the dimensions of the shaft, which have been mentioned elsewhere. The ground through which the shaft passes is somewhat more varied in character, as may be seen by reference to Fig. 9. The final grade, as originally contemplated, was somewhat lower than on the Arizona side, but the difficulties of construction were such that the final elevation of the cutting edge was fixed at 28 ± ; and the bottom of the shaft was sealed at that elevation.

Work Under Compressed Air.

When it became evident that, owing to the shattered and porous condition of the material surrounding the shaft, open tunnel methods were out of the question, a Consulting Board, consisting of Louis C. Hill, M. Am. Soc. C. E., Supervising Engineer; Francis L. Sellew, M. Am. Soc. C. E., Project Engineer, of the United States Reclamation Service; and Silas H. Woodard, M. Am. Soc. C. E., formerly Division Engineer on the East River Tunnels of the Pennsylvania Railroad. This Board made an investigation of the existing conditions, and advised a change in method to tunneling by the pneumatic process, the work to be carried on from the Arizona shaft only.

A compressor plant was rented from Charles A. Haskin and Company, Tunnel Contractors. Owing to the impossibility of securing suitable labor, skilled in such work, in that part of the country, experienced "sand-hogs" were brought from the East, to form a nucleus of trained compressed-air tunnel men. E. C. Hayden, Assoc. M. Am. Soc. C. E., was Superintendent of Construction on the work from this time until the successful completion of the tunnel. The rented equipment was as follows: Three Ingersoll compressors: 20 by 30-in., 24½ by 24-in., and 17 by 24-in; vertical and horizontal air locks and equipment, medical lock, air receivers and coolers, and the 4-ft. steel shafting used above the air deck.

The rated capacity of the compressor plant was 4 000 cu. ft. of free air per minute compressed to 40 lb. The maximum consumption for 24 hours averaged 2 000 cu. ft. per min., or about 55% of the rated capacity. The maximum recorded consumption for a short time

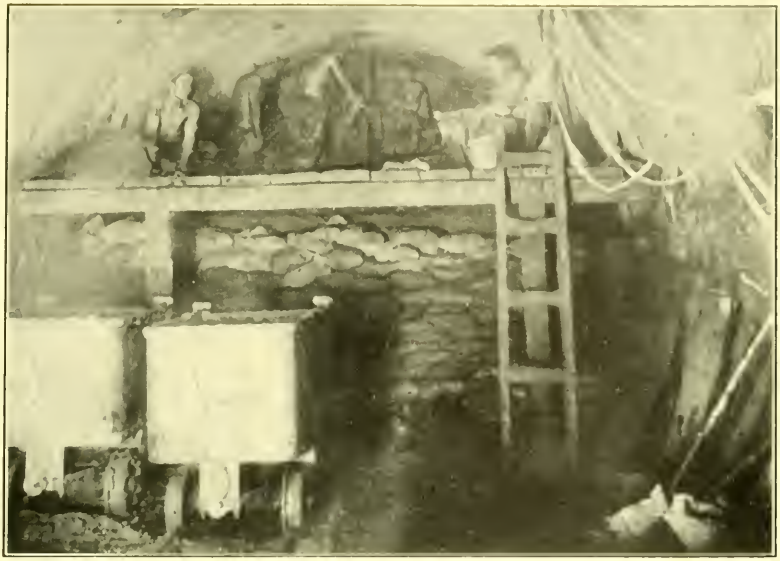


FIG. 1.—SAND-BAG BULKHEAD. EXCAVATING IN HEADING.

FIG. 2.—VIEW OF TUNNEL.

was 2 200 cu. ft. per min. The compressor plant was thus never seriously taxed for a long period. The average consumption during the tunneling operations was 1 450 cu. ft. of free air per minute, to a pressure of 28 lb. per sq. in.

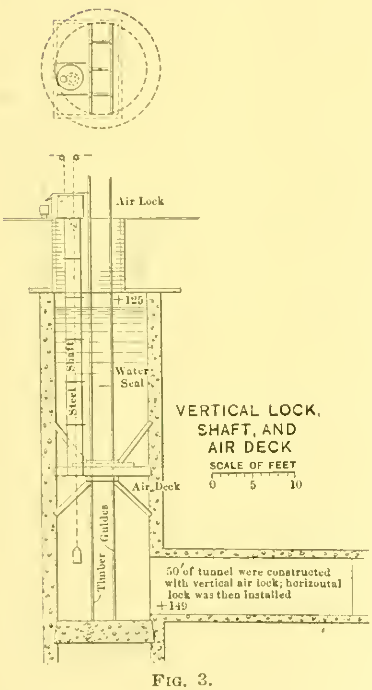


FIG. 3.

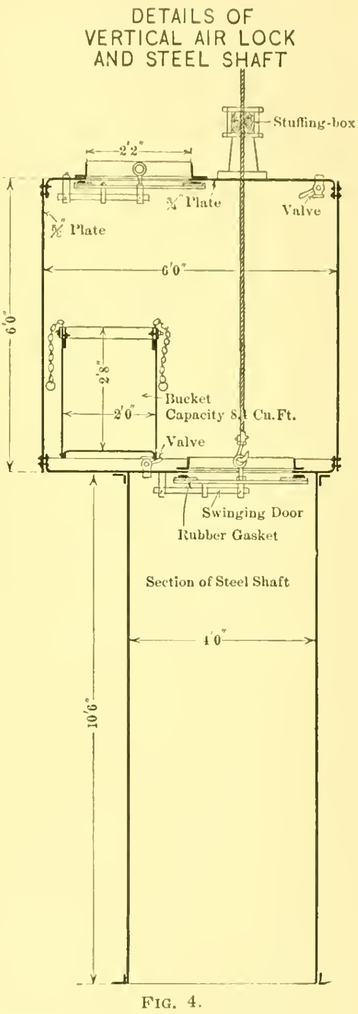


FIG. 4.

In addition to the foregoing compressor equipment there was already installed a 12¼ by 14-in. Ingersoll-Rand high-pressure machine, with a capacity of 200 cu. ft. per min. at 100 lb. This had been used during grouting operations at the Arizona shaft, and, during the further prosecution of the work, was used for operating drills and the

concrete lock, and also as emergency equipment at times when the consumption of air was great, or when it was not advisable to fill the air lock by withdrawing air from the heading.

Aside from the rented equipment, that which was already on the job was used. The steam plant consisted of five oil-burning, horizontal, tubular boilers of varying sizes, some of which had been used previously for the operation of mixers, hoists, derricks, etc., and others of which were available on other parts of the Project and were later installed when found necessary. The total rated capacity of the plant was about 475 boiler horse-power. This served for operating the compressors and all other machinery, and for heating the change and locker rooms. The fuel oil was stored in one concrete and one wooden tank of 720 bbl. total capacity, which had gravity feed to the steam plant, and were replenished by pumping directly from oil cars on a side track of the Southern Pacific Railroad. The derrick, which had been in use for excavation and installation work, and for replenishing the sand-and-stone hopper, was left in place, and was used later for handling the material for the outlet structure and for dismantling the plant. It was a 20-h.p., 7 by 20-in. American Hoist and Derrick Company derrick. For handling the muck and concrete, ½-cu. yd. steel end dump cars were used. These were of 24-in. gauge, 24 in. wide, 40 in. high above the rail, and were manufactured by the Pacific Coast Manufacturing Company and the Arthur Koppel Company.

In general, the preliminary installation was as follows: A timber air deck was built at mid-height of the shaft. From this a 4-ft. steel shaft led to the vertical air lock at the surface. Material and men were carried in cylindrical, 8-cu. ft., buckets. The air deck was built up of twelve layers of 2 by 12-in. yellow pine planks nailed with 40-d. nails, alternate layers running at right angles. A recess, 4 in. deep and 24 in. high, on the inside circumference of the shaft, provided bearings for the deck against upward and downward pressure. The deck was braced near the center in both directions by four 12 by 12-in. timbers at 45°, the ends bearing in recesses cut in the shaft walls. A 4-ft. circular hole was left, and the bottom cylinder was fastened with bolts passing through its flanges and the air deck. Other holes were left for pipes for air supply, electric light, and telephone wires.

From the deck to the surface was 72 ft. Six sections of 4-ft. cylindrical steel shaft were used. These were made up of ¼-in. steel plate with lap-riveted joints and 4 by 4 by ½-in. angle flanges at the top and bottom. Gaskets were used between sections, which were bolted together through the flanges.

The vertical air lock was a 6-ft. cylindrical drum 6 ft. high, with 2½-ft. circular plate doors in the top and bottom. The ends of the drum were of ¾-in. plate, dished and double-riveted to the side plate, which was ⅝ in. thick. The 8-cu. ft. buckets cleared the opening by about 1 in. all around. The cable for hoisting the buckets through the steel shaft passed through a gland in the top of the lock; it was of especially compact texture and smooth surface, thus reducing the leakage of air to a minimum. The capacity of the lock was about five buckets, or ten men, with the lock-tender.

A water seal was placed above the air deck, completely filling the shaft to Elevation 129. The top of the air deck was at Elevation 85. The actual maximum loads which the air deck was required to carry were (*a*) a pressure of 36 lb. per sq. in. on the lower side of the deck, with a 43-ft. water seal on the upper side, and (*b*) the weight of the 43-ft. water seal with no pressure on the lower side. The first condition gave a total net uplift on the deck of about 500 tons, which was transmitted to the shaft by the bracing timbers, and by the deck at its edge. The second condition gave a total unbalanced downward pressure of about 700 tons. As the deck was designed for somewhat more severe conditions, the actual stresses were low, and the timber gave but slight evidence of "working" while under full load. The under side of the deck was made air-tight and fire-proof with sheet asbestos tacked on and plastered over with clay-grout.

The boiler and compressor installation and the remainder of the above-ground construction plant were completed during this period. The hoist, mixer, derrick, and change room had been used on the previous work. Two units were added to the boiler plant, and the compressor plant and medical lock were set up.

During the construction of the air deck, the water level in the shaft had been kept down by pumps about to Elevation + 80. The air pressure was now raised and the water blown and pumped out. The relation between the river water surface and that in the shaft, with the air pressure, showed that the soft and porous material surrounding

and underlying the shaft offered small resistance to the transmission of hydrostatic pressure. When the water level had been lowered to + 42, the pressure was 36 lb. per sq. in. At that time the river water surface was at about 122. Elevation 42 was set as the bottom elevation of the concrete plug. Allowing for the thickness of the plug, and the sump, this brought the elevation of the flow line of the tunnel at + 49. The following considerations influenced the fixing of this grade:

First, the tunnel constructed from this elevation would have a minimum cover at high stage of the river of about 15 ft., which was considered sufficient for safety of construction in this material. As a matter of history, the tunnel did not reach the deepest point of the river until after the flood period, and the cover at that time was more than 30 ft.

Second, there was no assurance that the ground would be more favorable lower down.

Third, the difficulties, slowness, and expense of construction increase greatly with higher pressures.

The plug in the bottom of the shaft was designed as a circular reinforced concrete diaphragm, supported at the edge, to resist the maximum hydrostatic pressure, which at highest stage of the river would be about 40 lb. per sq. in. It seemed probable that the diaphragm would receive its full load while the concrete was still comparatively new. In order to make it possible to relieve the pressure somewhat, ten 2-in. pipes were set, to pass through the plug; the lower ends were nested in crushed rock to insure a free opening, and the upper ends were furnished with globe valves.

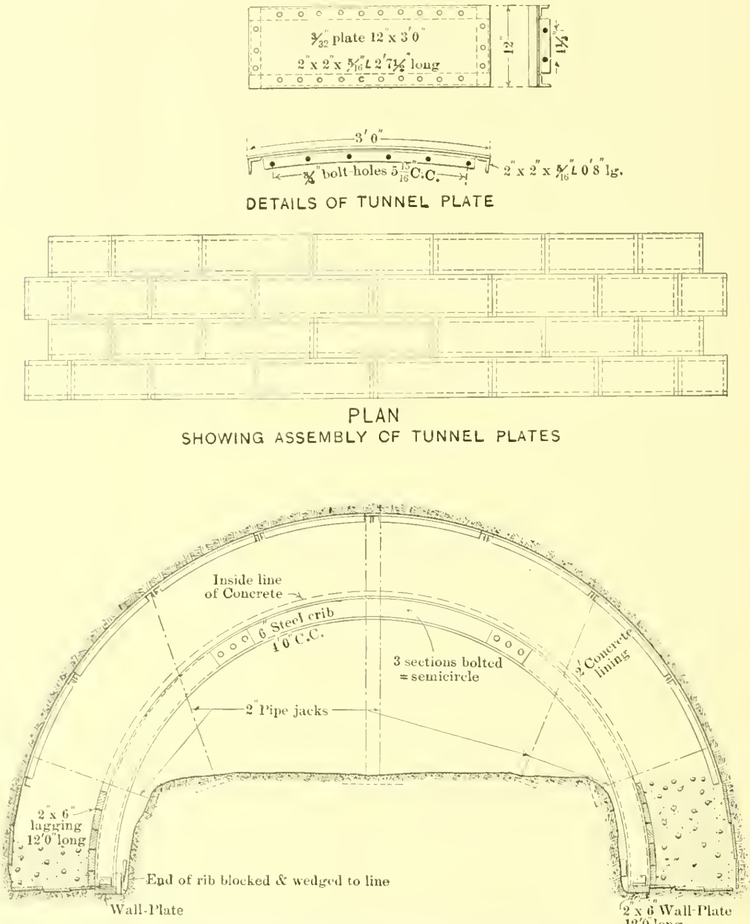
To provide a support in the shaft walls for the edge of the plug, a circumferential recess of the full height was cut, and the concrete was roughened to a depth varying from 3 in. at the top to 7 in. at the bottom. Any reinforcement encountered was cut and bent out to aid in bonding. The reinforcing steel for the plug consisted of $\frac{3}{4}$-in. and 1-in. square twisted rods, and old 15-lb. industrial rail. It was lowered into the shaft through an improvised pipe-lock. A 4-in. pipe was used, with a gate-valve at the top and a board flap-valve at the bottom. The bottom being capped, the steel was lowered from the top until its end rested on the board-valve. The top was then closed and the pressure equalized; the rods were dropped through and on a platform, and were then set in place. In this way steel was lowered as

fast as it could be set. Concrete was lowered in buckets through the 4-ft. steel shaft from the vertical air lock at the surface, and also through a small material lock. This consisted of a hopper closed at the bottom by a 4-in. quick-opening gate-valve, and at the top by a flat circular plate, 1 ft. in diameter, clamped down by a hand-wheel screw. In the top of the hopper there were: a pipe connecting with the compressor, a discharge valve, and a gauge. The capacity was about 3 cu. ft. The concrete was placed in the hopper, the top was clamped on, and the pressure was raised until it slightly exceeded that in the shaft. The lower gate-valve was then opened, and the concrete dropped through directly to the bottom of the shaft. Two men at the bottom shoveled the concrete away and handled the emergency board-stop at the end of the pipe. The operation of this lock proved unsatisfactory when there were bends in the pipes, and it was not used for tunnel concreting.

When the plug was completed, tunnel work was started. The $3\frac{1}{2}$-ft. shaft wall was cut out on the upper half of the tunnel, and was trimmed, roughened, and keyed so as to provide good bond with the first tunnel arch section. The ground immediately surrounding the shaft for a distance of about 5 ft. was found to be very bad; it had been shattered and softened by the shooting during the sinking of the caisson, and was saturated with water, practically of the consistency of quicksand. In places small "boulders" formed by the grout washed up from below were found. Where the grout pipes had been withdrawn, the material was consolidated not more than 2 in. outside of the original location of the pipe.

The heading was carried out as a 6-ft. high segment of the circular tunnel. To support the roof and sides, 12 by 36-in. steel plates, shown on Fig. 6, were used. These were made of $\frac{3}{32}$-in. steel plate, bent to a 9-ft. radius, with flanges at the four edges of 2 by 2 by $\frac{5}{16}$-in. angle-irons; the outstanding flanges were punched with $\frac{3}{4}$-in. holes spaced so that alternate rings could be staggered and bolted together with $\frac{5}{8}$-in. bolts. The breast was boarded and clayed tight. The ground was cut out on the circumference the size of one plate, and the plate set and bolted. The space between the plate and the roof was filled with clay. In this way a ring was completed from the crown to about 3 ft. above the springing line on each side. Trench braces were set radially on every second or third ring. The breast was then cut down level with the bench breast-boards, reset, and clayed, and the process

repeated. The joints between plates and between rings of plates were clayed over and mud-washed at intervals. Empty sacks and clay sacks were used for "blows."



CROSS-SECTION OF HEADING SHOWING
TUNNEL PLATES, BRACING, CONCRETE FORMS, ETC.

FIG. 5.

When an advance of fourteen rings beyond the finished work had been made, the heading, roof, and breast were tightened, the pressure was raised from 2 to 3 lb., and side trenches were cut to the springing line. Wall-plates were set to line and grade, steel forms set, and the 2-ft. concrete lining was placed. The forms were 6-in. channels bent

to a radius of 6 ft. 10 in., the 6-in. dimension being radial; they were in three sections for the semicircle, and fastened with ⅝-in. bolts. The lagging was 2 by 6-in. surfaced lumber, 12 ft. long. The longitudinal joint at the springing line battered from face to back in such a way as to facilitate grouting and caulking the joint on the completion of the invert concrete.

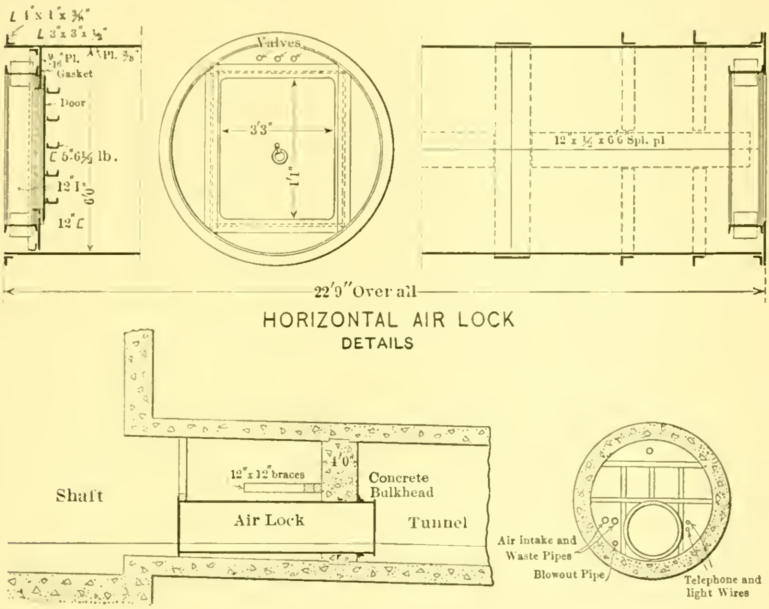


FIG. 6.

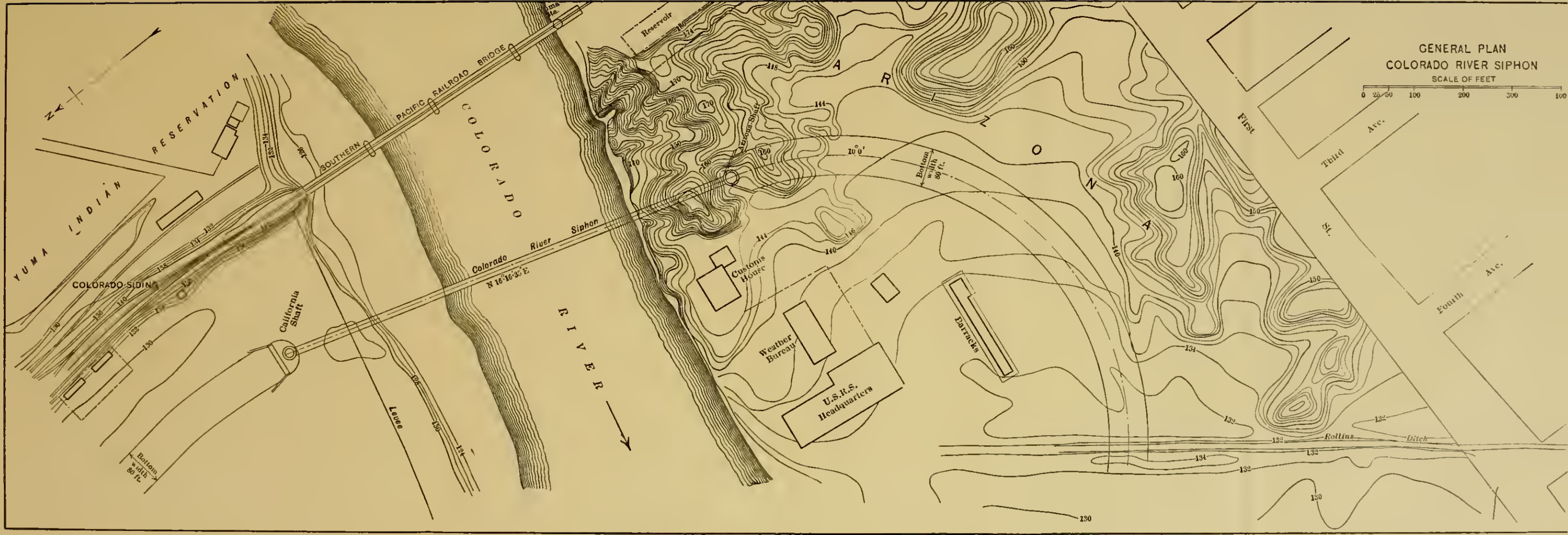
Fig. 6 shows a section of the heading completely excavated, with the forms and lagging set, ready for the concrete. The concrete was dumped from the buckets into barrows, wheeled to the heading, and shoveled into the forms. In this way the top heading was constructed for 55 ft. The pneumatic pressure was at all times practically equal to the hydrostatic head. The bench was then excavated in sections of from 10 to 12 ft. For this the pressure required was from 4 to 6 lb. higher than for the heading. Sections of bench lapped with sections of heading, so that there was no joint in the entire tunnel on one plane. The arch concrete carried itself on about a 10-ft. span longitudinally, during the concreting of a section of invert. In excavating the bottom, it was seldom necessary to brace the sides, except immediately

under the arch concrete. Here 16-ft. trench braces were set across the full width. The sand was kept plastered with clay to within a few feet of the bottom. When the excavation was completed, profile bulkheads were set at the end, to line and grade, and the bottom was filled with concrete to a level with the tops of the profiles. On the concrete there was then laid 12-ft. lagging, one end lying on the finished work of the previous section and the other on the profile; then the semicircular steel arch ribs, reversed, were set on the lagging and wedged and braced to line. This being done, the concrete and the lagging were carried up together to the joint with the completed arch. The joint was then grouted up under a small head.

On the completion of the first 55 ft. of tunnel, a bulkhead, $3\frac{1}{2}$ ft. thick, was built of brick laid in cement mortar, closing the heading. It was supported near the middle by four 12 by 12-in. diagonal struts bearing in the tunnel walls. After allowing the mortar a few days to set, the pressure was lowered to about 10 lb. Leaks and open joints in the tunnel concrete were marked, the pressure was raised, and the joints were cut out and caulked with cement mortar. After a suitable interval, the pressure was lowered to atmospheric, and preparations were made for putting in the horizontal air lock. All leaks in the shaft had also been thoroughly caulked, and there was now practically no leakage. The water seal in the shaft was pumped out, the steel shaft, vertical lock, and air deck were removed, and hoist cages were placed.

The tunnel air lock was a 6-ft. cylinder, 22 ft. 9 in. long. It was of $\frac{3}{8}$-in. iron plate with double-riveted, butt splices; the door-frames were built up of 12-in. channels and I-beams, and the doors were of $\frac{1}{2}$-in. plate, reinforced with five 5-in. channels. On the outside near the pressure end there was a collar, made of two 3-in. angles, to provide bearing against and bond with the lock bulkhead. The air pipes and valves were hung with stirrups from the top of the lock. The filling or discharge of the lock could be regulated from either end, there being a double set of 2-in. supply and discharge valves. The capacity of the lock was five $\frac{1}{2}$-cu. yd. cars.

The bulkhead was of plain $1:2\frac{1}{2}:5$ concrete, 4 ft. thick, with two sets of diagonal braces against the tunnel walls. Bearing on the circumference was provided by a 3-in. recess which had been left in the concrete lining on this section. Pipes for air supply, telephone and electric light wires, and drainage were concreted in.


GENERAL PLAN
COLORADO RIVER SIPHON
SCALE OF FEET
YUMA INDIAN RESERVATION
COLORADO SIDING
SOUTHERN PACIFIC RAILROAD BRIDGE
COLORADO RIVER
Colorado River Siphon
N 16°16'35" E
California Shaft
Levee
Reservoir
ARIZONA
Customs House
Weather Bureau
U.S.R.S. Headquarters
Barracks
First St.
Third Ave.
Fourth Ave.
Rollins Ditch
Bottom width 80 ft.

When this work had been completed (October 20th, 1911), the pressure was again raised and the temporary brick bulkhead was cut out. The method of advancing the heading has already been outlined, and in general this method was adhered to throughout the construction. Sandbag and clay bulkheads were placed in the lower half of the tunnel at the end of the completed work during the advance of the heading, the pressure usually varying from 3 to 5 lb. on the heading and bench excavation. Grouting pipes with valves were placed in the crown at the end of each 12-ft. section. The pressure was at all times within a few pounds of the hydrostatic head, the lower average pressure during the winter reflecting the lower stage of the river.

The heading was always kept tight by plastering with clay, and there were no serious blows of long duration. While the air chamber in the tunnel was small, the air lock was filled by a direct connection with the auxiliary compressor, to avoid variations in pressure caused by locking in directly from the heading.

Fig. 7 shows the geological formation through which the siphon passed. As has already been stated, the material surrounding the shaft softened during the sinking of the caisson. Within 15 or 20 ft. it became somewhat harder, but was very porous, and crumbled into loose sand when excavated. A pocket of loose gravel in the lower portion of the top heading ran from Stations 3 to 5. On this section it was necessary to carry the tunnel plates or poling boards to the springing line, and to brace the side trenches against the bench thoroughly. In excavating the bench, the upper portion of the sides was planked, and long trench braces were placed horizontally across the tunnel about every 4 ft. From Stations 5 to 7 a pocket of large boulders, with pea gravel between, interfered somewhat with progress. The gravel, once started, ran like small shot, and work progressed slowly, owing to the danger of a slump in the roof or breast. A lense of exceedingly hard and thoroughly cemented gravel, or puddingstone, was encountered between Stations 5 and 6. The last 300 ft. brought to light various pockets, large and small, of boulders, clay, gravel, and quicksand. Both experienced prospectors and amateur miners were disappointed in not finding gold-bearing gravel under the Colorado, although an incipient boom was started by salting a car of muck with brass filings.

From Station 7 to the California shaft the material was somewhat more favorable, and progress was steady. On April 12th a galvanized-iron well casing was encountered. This was one of the original test

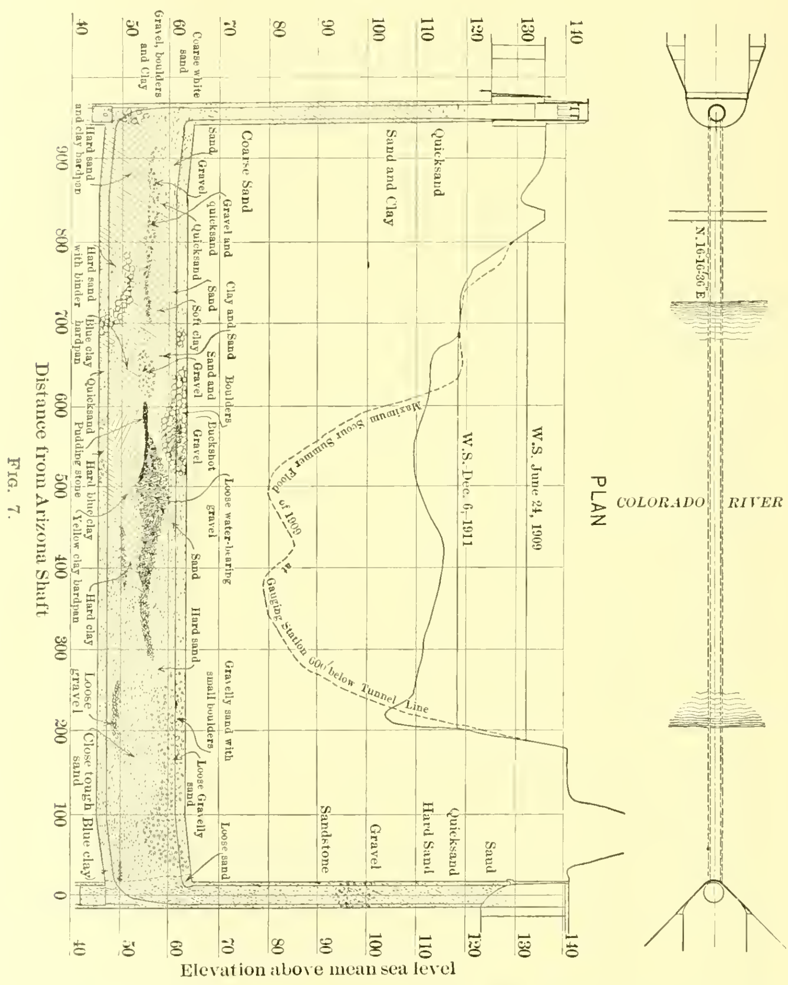


Fig. 7.

wells sunk before the work was started. On May 2d the California shaft was reached. The concrete was roughened and grooved and the tunnel concrete well bonded to it. The tracks, platforms, and muck were then cleaned out, and the rough places in the invert finished off.

The pressure was dropped to 5 lb., and all leaks, cracks, and porous spots were marked; then the pressure was raised to 30 lb., and the leaks were caulked. The tunnel was then opened for inspection. The leaks remaining were insignificant, and the work all appeared to be in good condition. The lock bulkhead, lock, and cage guides were removed, and the quarter turn in the bottom of the Arizona shaft, described elsewhere, was constructed.

From the time of the installation of the horizontal lock to time of completion of the tunnel, the average progress was 4½ ft. per day. The best monthly progress was 165 ft., in favorable ground, during January, 1912. The slowest progress was during February, in gravel, boulders, and sand; in this month a length of 100 ft. of tunnel was completed.

Alignment.

The elevation of the flow-line at the shafts is + 49. For 100 ft. from each shaft the grade is 2% down, and between these points the bottom is level. The line is straight between shafts.

On the surface a parallel offset line was run, there being obstructions on the direct line. After pumping out the Arizona shaft, and previous to the construction of the air deck, the surface line was plumbed down and plugs were set in the shaft walls. After the installation of the horizontal air lock, the surface line was brought down and carried through the lock four times, to eliminate errors. The detailed operation was as follows:

The transitman, setting up on the surface line, directed the placing of two 2 by 3-in. wrought-iron staples, on each side of the shaft, the line falling within the cross-bar of each staple. These were driven into the solid crib timbers inclined over the shaft, at such an angle that the plumb line, hanging over them, would clear all obstructions below. The line was marked on the bar with a file, and the steel wire plumb lines, carrying 20-lb. plumb-bobs, were placed in the file marks. The transitman, by moving the telescope only slightly, or not at all, in a vertical direction, could then observe both wires directly below the points of support, sighting between the legs and over the top of the nearer staple. The plumb-bobs were hung in pails of water. The length of the base line thus obtained was 10 ft. 6 in.

A 4 by 6-in. timber was placed across the tunnel end of the air lock, and securely wedged and braced. The transit, on a small trivet,

was set on this timber and lined in by sighting on the plumb wires, oiled paper and a candle being used for illumination. A scratch was made on line on the far end of the lock, above the door. A second transitman stayed in the shaft to observe the lock for motion due to jarring or working while the pressure was raised. The first transitman then locked through, checked his setting on the mark over the lock door, plunged his transit and took readings on two scales fixed in the tunnel roof at 30 and 100 ft. from the lock. Without lifting the transit, the operator again locked out and checked on the position of the plumb wires in the shaft. If the check was unsatisfactory, the line was again locked through and back. The transit was once more set up on the surface and the position of the wires was checked back to the original line, thus completing the circuit. While the alignment work was being done, one of the two cages was out of commission, the plumb lines occupying the one cage well. The work was done at a time when all material could easily be handled with one cage.

The level-work differed in no way from surface leveling, a point in the lock being used as a turning point. At each setting of the forms, a grade nail was set in the breast.

In carrying forward the line as the work in the tunnel progressed, two 2-in. wrought-iron staples were driven into the roof at the end of every 60-ft. section, on line and about 15 ft. apart. The exact line was marked with a file on the cross-bar of each staple, and a plumb line was hung in the file mark. The work of setting the forms preceding each run of concrete was done with a rule, a string, a plumb-bob, and a carpenter's level.

The string was stretched to the breast, tangent to plumb-lines hung in the file marks, and nails were set in the breast and at the end of the completed work. Between these was stretched a line, from which the foreman afterward could check the setting. Measuring from this line, the wall-plates were set in the side trenches, exactly to line, and approximately to grade. The first form was brought in, bolted up, and set nearest the breast, the ends, which were on the horizontal diameter, being braced securely to line, as given by the wall-plates on which they rested. The grade was then checked from the nail in the breast, and the rib was shimmed up to the proper level. The intermediate ribs were then set up and quickly aligned between the pilot form and the completed concrete. The cross-section was then measured

up, and what slight trimming and shaping remained was done in a short time. The concrete of the preceding section was cleaned and roughened, and the new run started.

The setting of the forms for the invert was somewhat simpler. A profile-bulkhead was constructed, made up of three thicknesses of 1-in. planks, the radii of the upper and lower edges of the profile corresponding to the inner and outward radii of the concrete lining. It was cut on radial lines into interchangeable sections 3 ft. long, the edges being bound with iron and carefully trued. When the excavation of a 10-ft. length of invert was complete, the profile was set up at the end of the section, the line and grade being measured from the completed concrete of the arch. The bulkhead was secured on the concrete face by iron pins driven into the ground, and was wedged and braced against the bench. As has been previously outlined, the bottom was filled with concrete; 12-ft. lagging was then laid on the concrete, one end resting on the completed work and the other end on the profile. The arch ribs, reversed, were set on the lagging, and secured against uplift by wedges and a cross-brace. The lagging and the concrete were then carried up together. The same set of profiles was used throughout the work. It was impossible to check the line and grade at the California shaft until the tunnel lining was complete and sealed to the shaft. The differences, however, were inappreciable, and the adjustment was made in the 3½-ft. thickness of the shaft lining.

Caisson Disease.

Doctors Henri ApJohn and O. I. Tower were medical examiners for the Reclamation Service. A certificate signed by the examiners was required before a man was employed at tunnel work.

The proportion of green men was necessarily very high, as there has been no compressed air work of any magnitude in the region supplying Yuma with labor. About twenty seasoned men, including the foremen, were brought from the East, and this group remained throughout the work. The remainder of the force shifted constantly; it was composed largely of Mexicans and "floaters" who came to spend the winter in Yuma. The conditions, therefore, were unfavorable, as far as the physical make-up and the seasoning of the laborers were concerned.

At the start, when the pressure rose above 30 lb., it was found

necessary to cut down the time spent under compressed air, so that the gangs worked a total of 4 hours in the heading during an 8-hour shift. The larger number of cases of "bends" at the start, under conditions apparently favorable as to air supply, was undoubtedly due to the fact that the highest pressures, averaging more than 35 lb., came at a time when the majority of the force was composed of green men. However, with the short shifts, there were no fatal cases. Later, although the pressure was at times as low as 27 lb., the same hours were adhered to, it being impracticable to vary the schedules with the varying pressures.

The time of locking out was fixed at from 12 to 15 min., the rate of decompression being made constant. This rule was rarely violated, except by the lock tenders, who were in and out at very short intervals, and whose tissues were never "saturated" by air under pressure. A medical lock, similar to those used on the East River Tunnels, was used for recompression and treatment of cases of "bends."

The total number of different men employed on compressed air work, from start to finish, was about 1 300. All these men had at one time to pass the medical examination. Some who were passed at first, but who were very susceptible, were eliminated later. However, in those cases in which the symptoms were merely pains in the extremities, which were relieved by recompression or local treatment, the men were at liberty to return to work at their pleasure. As the work was done in a small town, and the men lived at the Government bunk-house or at the other rooming houses close at hand, there were probably no serious cases which escaped the notice of the physicians. The records show the following cases treated:

(1) Cases causing disability for a few hours.......	299
(2) Cases causing disability from one to three days.	38
(3) Cases of partial paralysis, including bladder and lower extremities, causing disability for a few weeks	4
(4) Cases of total paralysis, resulting in death in three weeks................................	1

Those in Groups (1) and (2) were largely cases of pains in the arms or legs, with a few cases of vertigo. These were numerous at the start; the most susceptible men, having suffered once, were usually

willing to leave the work after the experience; these cases, therefore, became of less account as the work progressed. The cases in Group (3) were not traceable to extraordinary conditions on the work at the time of their occurrence. They were scattered, and did not coincide with any noticeable increase in the other cases, and must be laid to unknown factors in the condition of the men at the time. The one death was of a seasoned man, who had passed the medical test and had worked for some time. His attack of paralysis followed a prolonged period of dissipation which left him in poor health and unable to withstand conditions which he might otherwise have weathered successfully. The record of only one death in the large number of men employed, and that not directly chargeable to conditions on the work, in a job which lasted for 10 months, is fairly satisfactory, as compared with other records.

The experience at the Colorado Siphon probably adds nothing new to what is known of the contributing causes and the treament of caisson disease. These have been fully discussed in recent papers presented to the Society or published in engineering journals.

California Shaft—Intake Structure.

The condition at the California shaft at the commencement of pneumatic work has already been described. The work remaining to be done at that side of the river was, first, the construction of a plug, at the same elevation as on the other side; second, arrangements for making the junction from the tunnel end and cutting through the shaft wall without loss of pressure; and, third, the construction of an intake structure.

The plug requires only brief mention. It was of reinforced concrete, 4 ft. thick, and 17 ft. in diameter; and was bonded and keyed to the shaft wall, as on the Arizona side.

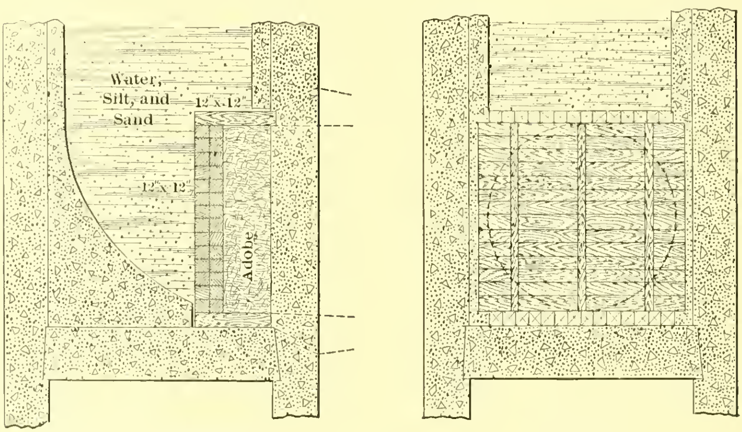
Above this there was constructed a quarter turn in the shaft, to change the direction of flow of the water from vertical to horizontal. This was of concrete, and built on a center line radius of 8.5 ft. Above the quarter turn there was a reducer, 15 ft. long, from the shaft diameter of 17 ft. to the tunnel diameter of 14 ft.

The tunnel opening into the shaft was to be between Elevations 49 and 63. A vertical timber bulkhead, of two thicknesses of 12 by 12-in. timbers, bolted and keyed together, was built across the proposed

opening. The ends of the timbers were recessed into the concrete, and a flooring and roofing of 12 by 12-in. timbers placed. The space was filled with adobe. In this way, when the tunnel was cut through, it would open into a compartment entirely separated from the main shaft by a pressure-tight bulkhead, and a strong, water-tight joint could be made. The shaft outside of the bulkhead was filled with sand and water to Elevation 120.

During the construction of the tunnel the work on the intake structure was completed. This, together with the main check and wasteway, briefly described later, controls the flow of water through the upper portion of the main canal in Yuma Valley. As may be seen

BULKHEAD IN CALIFORNIA SHAFT



SECTION ON CENTER LINE

ELEVATION

FIG. 8.

by referring to the plan, Plate XLVI, it consists of a covered concrete basin surrounding the shaft, and a cylindrical gate and operating machinery by which the basin may be cut off from the shaft. As it is of somewhat original design, a more detailed description may be of value.

The main canal at this point is of trapezoidal section, having a bottom width of 80 ft. and a water depth of 7 ft. Warped retaining walls reduce this to a rectangular section 72 ft. wide and 7 ft. deep. From a line 15 ft. in front of the center line of the shaft, these walls, 11 ft. high, swing in, and around the shaft on curves of 36 ft. radius, compounded with curves of 17 ft. radius. At the rear of the shaft,

where these walls meet, and at points 120° from it, are constructed three piers, enclosing the guides for the cylinder gate. The floor of the basin is 1 ft. thick, with the top at Elevation 125, which is also the elevation of the canal bottom. The flow of water is directly down into the shaft, there being no obstruction except the guide piers. The whole basin is roofed over with a reinforced concrete girder floor, which is on a level with the top of the canal banks. On this floor is built the operating tower, into which the gate is drawn when the siphon is in operation, and which carries the operating machinery.

The gate has been described as a "bottomless tin can", set over the top of the shaft; it is 8 ft. high and 21 ft. in diameter. When it is down on the sill, its top is 1 ft. above the normal water level in the canal. The skin plate is $\frac{3}{8}$ in. thick, stiffened at the top and bottom by rings of 3 by $3\frac{1}{2}$ by $\frac{3}{8}$-in. angles, which in turn are supported by 3 by $3\frac{1}{2}$ by $\frac{7}{16}$-in. angles as chords. The whole is held rigidly to a circular shape by three radial stiffening or tie-brace trusses, at 120° from each other, connected to a cast-steel, three-winged center post. The gate-stems are secured to the gate at these three points on the circumference, as are also the guide-bearing castings.

There are two types of guides, designated *A* and *B*. The two Type *A* guides merely present a free bearing surface, which constrains the gate only in a radial direction. Type *B* guide, which is embedded in the heavy rear pier, encloses a portion of the bearing casting on the gate, and thus prevents both radial and circumferential motion. All three guides are single castings 19 ft. 5 in. long, the lower 2 ft. being embedded in the foundation. They are anchored in the concrete piers throughout their length, and are fixed at the top by a triangular lattice girder frame which is embedded within the girders of the operating-tower floor. The guides and bearings are of cast iron, with machined wearing faces.

The circular cast-iron sill on which the gate rests is shown in section with the gate on Plate XLVIII. It is made in six sections, bolted together, and anchored in the concrete. It is bolted to the guides at the three points of contact. In placing the sill, the shaft was trimmed out roughly to form a bed, and the sill was brought to grade with iron shims, and grouted in place.

As previously stated, the cover of the basin is a slab and girder floor. It is surrounded on all sides with a 2-in. iron-pipe railing.

Above it is the operating tower, of cylindrical form, 8 ft. 8¾ in. high, which carries the gate-stand and operating machinery, and also forms a sleeve into which the gate may be drawn. The three gate-stands, on two shafts at right angles to each other, are operated through a gear drive by a 3-h.p. continuous current motor. The gate weighs about 8 tons, and in ordinary operation is raised and lowered at the rate of about 2 ft. per min. The tower is accessible from the floor of the structure by a concrete stairway. Four cast-iron lamp-posts on concrete pedestals are placed around the structure. One of the main traveled roads of the Yuma Indian Reservation passes around the end of the canal at this point.

The construction of the intake was carried on with the same plant as that used in building the shaft. The foundations were excavated by Fresno scrapers and by hand. The foundation and floor were reinforced with old rails. The proportions of the concrete for the foundations were 1 : 3 : 6, and for the remainder of the structure, 1 : 2 : 4, granite screenings and sand being used for the finer material. The sand and stone were shoveled directly from the stock platform into the measuring hopper of the mixer; and the concrete was dumped into a 1-cu. yd. tip-bucket and conveyed to the forms by the derrick. Steam for the mixer and derrick was furnished by a 30-h.p., vertical, tubular, wood-fired boiler. The face forms for the warped walls were built on the ground in one section, 11 by 20 ft., of 1 by 6-in. tongued and grooved lumber, with studs about 3 ft. apart. They were lifted and carried to place by the derrick, and sprung to line and braced. Cut-off walls were carried 4 ft. into the ground, at the ends of the walls and at the edge of the floor.

The intake was completed in February. No additional work was necessary at the California shaft, except to remove the water and sand filling, and the temporary wooden bulkhead, described previously, and to make the connection with the tunnel.

Fig. 1, Plate XLVII, shows the placing of the cylinder gate on the shaft, and Fig. 2, Plate XLVII, shows the completed structure before water was turned into the main canal.

Outlet Structure.

The outlet structure is of much simpler design, there being no control of the flow at this point. The construction, however, was more difficult, owing to the topography, the want of space, interference

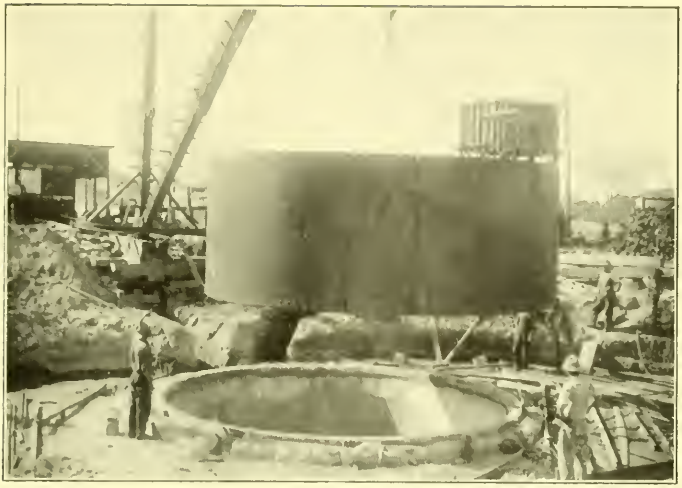


FIG. 1.—INTAKE STRUCTURE. PLACING CYLINDER GATE.

FIG. 2.—INTAKE STRUCTURE COMPLETED.

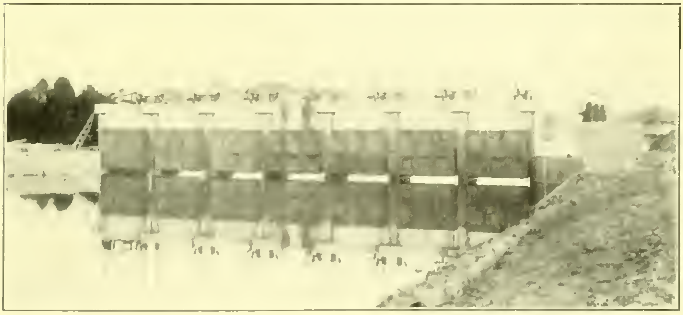


FIG. 3.—MAIN CANAL. CHECK AND WASTEWAY.

with the tunnel construction and the plant, and the rise in ground-water at the high stage of the river. The ground surrounding the Arizona shaft and for some distance from the river, is high and irregular in contour. The construction plant was grouped around the shaft in a basin at Elevation 147, which is 24 ft. above the bottom grade of the canal. The problem was to construct the retaining walls without interfering with the operation of the compressors, mixer, hoist, and boiler plant. The diagram, Fig. 9, showing the layout around the shaft and the outlet, will make clear the method of construction, which was to build the walls and foundation in a trench, the core being removed later.

The east wall was built in two sections, numbered I and II on Fig. 9. Each section was about 30 ft. long and 15 ft. wide. The bottom of the foundation grade was at Elevation 116.5 in I and at 118.5 in II. Sheeting, of 2 by 12-in. planks 20 ft. long, was driven as the excavation proceeded, with 6 by 8-in. rangers and cross-braces in horizontal bents about 5 ft. from center to center. Ground-water was encountered at Elevation 120; the soil was soft sand and tended toward quicksand below ground-water level. A small steam pump set in a sump at one end of the trench kept it dry. At Elevation 127 a second set of sheet-piling was driven. When subgrade was reached, a layer of crushed rock was placed, to provide drainageway and prevent the bleeding out of the sand under the wall by the flow of the ground-water. The reinforcing steel for the wall and foundation was then thoroughly wired in place, and the foundation concrete placed. Forms for the wall were built in 5-ft. lifts, the trench bracing being removed and back-fill being placed as the wall was carried up. The track from the mixer to the shaft was carried over the trench on stringers, and the track to the dump was moved to the other side of the shaft. The concrete for the wall was chuted directly from the mixer to the forms.

The full length of Trench III was opened at once; the trench bracing and sheeting was similar to that in the others. At this time the spring floods of the river had started, and the accompanying rise in ground-water was more than 5 ft. It was found impossible to carry the subgrade as low as on the opposite wall, as the upward flow of water through the ground grew too large in volume to handle conveniently and threatened to "blow up" the soil under the ends of the sheeting. The subgrade, therefore, was made at Elevation 119,

and the foundation and walls were constructed as on the opposite side. As this subgrade is 4 ft. below the flow line of the canal, there is small likelihood that the water will scour to this depth. The flow of water through the siphon during this irrigating season has been only a small portion of the full capacity of the structure, and the velocity at the outlet has been relatively low. During the present winter season a concrete floor will be placed on the canal bottom at the outlet, protecting the foundations against scour at full flow.

The retaining walls are designed to resist an uplift due to a 6-ft. head of water, combined with a pressure on the back of wall due to an angle of repose of 17° 30′. The wall is 3 ft. thick at the base, and has a batter on the back of 1 in 12. The is at Elevation 141. From the top of the wall the ground slopes back at a 2:1 slope to Elevation 150. There is a berm 16 ft. wide for a road, and back of it a gravel and sand hill.

The work remaining to be done from the Arizona shaft was the removal of the tunnel floor, the lock, and the bulkhead, the cleaning up of the tunnel, the removal of the cage-guides, gallows frame, and cribbing from the shaft, and the construction of a quarter-turn gooseneck at the bottom of the shaft, enlarging the diameter of the tunnel to that of the shaft.

The shape and dimensions of the gooseneck are shown on Plate XLVIII. The changes in direction and velocity are easy, and there will probably be no great loss of head at full capacity. Circular forms were built on radial lines, and the lagging was brought up and the concrete placed in 3-ft. or 4-ft. lifts. The last work done before the running of water was the removal of these forms.

On the ground between the retaining walls, where the canal leaves the shaft, there were the hoist, boiler plant, one small compressor, and the change room. The hoist was removed, and a small hoist was set temporarily on the opposite side of the shaft. The boiler plant was needed only for the derrick and hoist, as at this time the pressure was being lowered in the tunnel, and the compressors had stopped. The office was removed, and a temporary installation of three boilers was made at this point, and everything was removed from the canal right of way. The excavation of the canal had been proceeding from the other end of the heavy cut section with Fresnos and wheelers, and the short piece remaining between the completed portion and the shaft

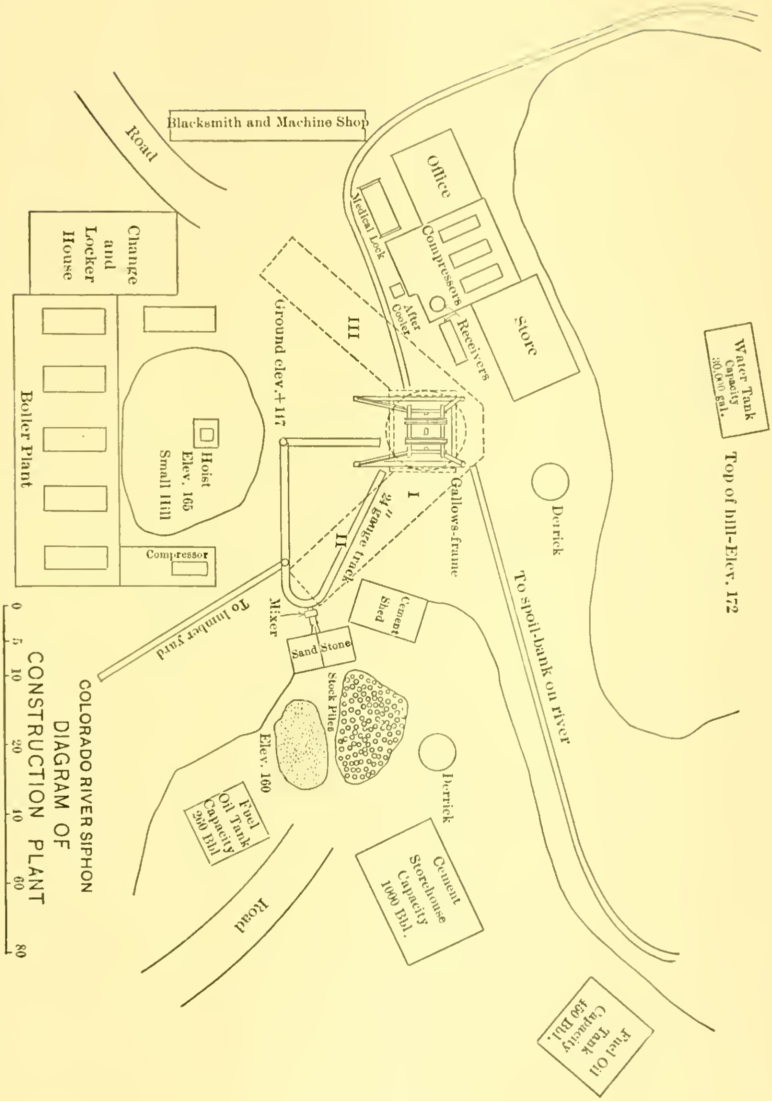

COLORADO RIVER SIPHON
DIAGRAM OF
CONSTRUCTION PLANT
SCALE OF FEET
0 5 10 20 40 60 80
Blacksmith and Machine Shop
Road
Change and Locker House
Boiler Plant
Hoist
Elev. 165
Small Hill
Compressor
Ground elev.+117
III
II
I
Office
Medical Lock
Compressors
After Cooler
Receivers
Store
Water Tank Capacity 30,000 gal.
Top of hill-Elev. 172
Gallows-frame
24" gauge track
Derrick
To spoil-bank on river
Cement Shed
To lumber yard
Mixer
Sand
Stone
Stock Piles
Elev. 160
Fuel Oil Tank Capacity 260 Bbl
Derrick
Cement Storehouse Capacity 1000 Bbl.
Road
Fuel Oil Tank Capacity 450 Bbl.

FIG. 9.

was now removed. Owing to the rise in ground-water it was impossible to excavate lower than 2 ft. above the final bottom grade of the canal, but a channel sufficiently large to carry all the water required this season was cut through.

The flow of water in the siphon may be controlled at the California shaft, or, in case of accident to this structure, at the main check and wasteway, ½ mile up stream. This structure, Fig. 3, Plate XLVII, consists of a concrete check with seven steel buckle-plate gates, and a concrete turnout and waste channel capable of taking the full flow of the canal. The waste channel leads to the Colorado River. The gates of the check and wasteway may be operated by hand or by electrical power.

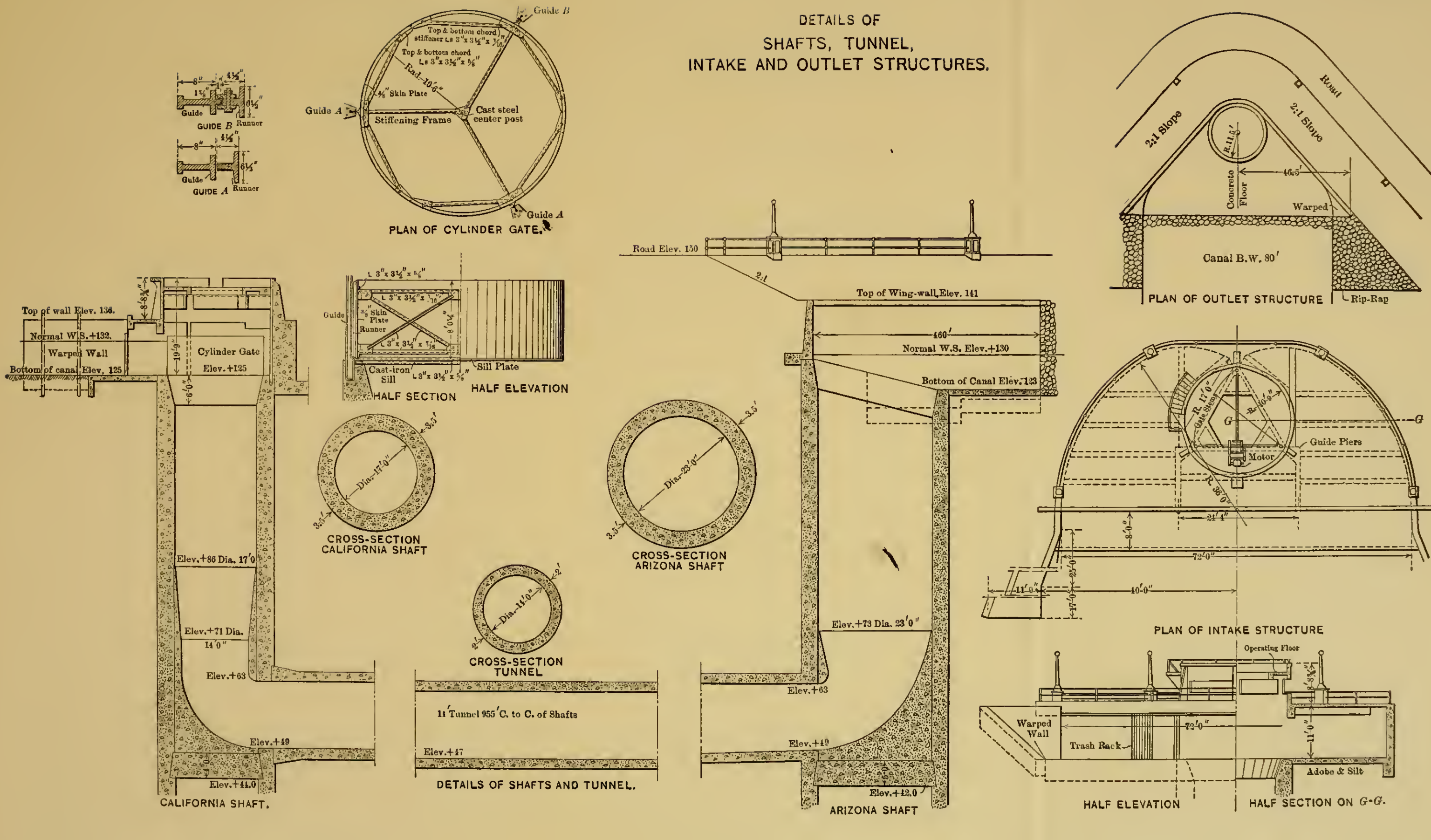
Water was turned through on June 30th, 1912. The consumption of water in Yuma Valley has not approached the maximum capacity of the siphon and will not for several years to come. At present, the flow is about 300 sec-ft., of which about 100 are wasted to the river on the Arizona side. The velocity in the shafts and tunnel, therefore, is low, being slightly greater than the silting velocity. Soundings taken in the shafts indicate that little or no silting has taken place. The cylinder gate is ordinarily kept raised about 6 in. from the sill. The velocity of the water between the gate and the sill, therefore, is about 9 ft. per sec.

The completion of the siphon makes available for irrigation by gravity the upper portion of the Colorado River delta, one of the most fertile tracts of land in the United States. The canal system in the Yuma Valley is about 65% completed; it comprises 35 miles of main canal and 120 miles of laterals. The system has been in successful operation during the 1912 irrigation season.

Francis L. Sellew, M. Am. Soc. C. E., has been Project Engineer of the Yuma Project since 1906. During this period Laguna Dam, the levee system, the system of main canals and laterals, and the Colorado River Siphon have been constructed.

The work on the shafts, previous to the commencement of pneumatic work, was under the superintendence of Messrs. R. P. Marable and F. W. Hall, with Mr. R. M. Priest as Assistant Engineer. F. Teichman, M. Am. Soc. C. E., designed the intake structure and the cylindrical gate. On the tunnel construction work, E. C. Hayden, Assoc. M. Am. Soc. C. E., was Superintendent of Construction, and the writer was Assistant Engineer.

DETAILS OF SHAFTS, TUNNEL, INTAKE AND OUTLET STRUCTURES.



AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

A BRIEF DESCRIPTION OF A MODERN STREET RAILWAY TRACK CONSTRUCTION.

Discussion.*

By A. C. Polk, Assoc. M. Am. Soc. C. E.†

A. C. Polk, Assoc. M. Am. Soc. C. E. (by letter).—The point brought up by Mr. Tratman, with reference to the use of ordinary bricks tilted so that one end fits under the rail head, is noted. This method gave very poor results on some track which had already been placed in Springfield; in fact, a great deal of the work was done in this manner previous to the time when the writer commenced work there. It was found that teams and loaded wagons had great difficulty in turning out from in front of cars, and a great many wheels of vehicles were broken. The flangeway, as formed in the Springfield work, will, no doubt, wear down somewhat on the edges of the bricks next to the rail, but on one piece of work which was in use 1½ years there was no serious trouble, and no undue amount of wearing had taken place. In practically every type of construction the wear is greatest at this point, no matter how the paving is placed. Mr. Polk.

The crowning of the pavement, as criticised by Mr. Tratman, although making a slightly uneven contour in the street, does not seem to cause any trouble with traffic; wagons and vehicles are able to turn out without any difficulty, and the uneven contour is scarcely noticeable. The city authorities and the public both thought this construction a great improvement over the old type of placing the brick under the rail head, and were very much pleased with it.

The type of construction described by Mr. Howe, namely, 6 in. of ballast under the ties and then concrete on top as a paving foundation,

* Continued from February, 1913, *Proceedings*.

† Author's closure.

Mr. Polk. was used by the writer in Mobile in placing a very busy cross-over. Reports indicate that it stood up very well under the constant and heavy traffic, and the writer has no doubt that, as a cheaper method of construction, this type answers very well, but he would not consider it as of a permanent nature.

The decision to put in the type used in the Springfield work was arrived at from previous experience with cheaper types, where it was shown that in the long run they were far more costly than a permanent and rigid construction.

One thing to be noted in connection with the Springfield work was the adoption of a standard section of rail of the Am. Soc. C. E. pattern. This, of course, made the cost less than it would have been with a girder rail.

Mr. Howe's method, as used in Oakland, seems to the writer to be an excellent compromise, where it is not possible to put in a more expensive type.

It is noted that Mr. Mitchell has had very poor results with the concrete beam construction. The writer, however, has had quite the contrary experience where the concrete beam has been placed and has been reinforced with twisted steel rods, as in the type described.

The rail ends were not ground on the Springfield work. Although, no doubt, this is a most excellent additional precaution, there was no visible sign of any movement at the joints, after the first work at Springfield had been down 12 or 14 months. As a matter of fact, it was very difficult, indeed, to discover the joints at all.

All the bolts, as mentioned by Mr. Mitchell, were driven to an extremely tight fit. There was no trouble from cupping on any work of this class, some of it having been in service more than two years, not only at Springfield, but at other places. A great deal of the strength and rigidity of the Springfield joints was due to the type of plate used.

It is noted that Mr. Vorce also has had poor results with the beam type of construction, but he does not state whether he used a reinforced beam. The writer believes that this reinforcing has a greal deal to do with the character of the beam construction, and makes it a decidedly different proposition from a beam which has no reinforcement. There is no doubt that poor foundations would affect the failure of the beam, but reinforcement, properly placed and in sufficient quantity, would hold where a straight concrete beam of larger section would fail.

The writer does not agree entirely with Mr. Vorce in his statement that a hard, rigid, inelastic track is undesirable. As a matter of fact, as soon as a track foundation is put down, in a street paving in which there is any movement at all, the surrounding pavement commences to work, and in a very short time will be broken up and destroyed. The writer is inclined to believe that, in building in an expensive and costly pavement where there is considerable traffic, and

where rebuilding is almost prohibitive, it is far cheaper in the long run to put down a solid, heavy roadbed. Mr. Polk.

The wooden ties would probably be satisfactory, but the writer is of the opinion that, if they are to be used in a permanent structure, it would be desirable to place tie-plates on them, which would, no doubt, add to the life and reinforcement of the track.

The writer has mentioned the criticism as to the grinding of the rail ends, and although it may be that the matter is of more vital importance than he considers it, his experience on two previous construction jobs, of the character illustrated in the paper, did not show that planing off the rail was nearly as essential as the proper placing of the joints and of the concrete around the ties, with a big, heavy tie under the joint, also the additional reinforcement at this point.

Mr. Vorce's description of the work in Vancouver is certainly most interesting, and should be of considerable value to any one interested in this particular type of work.

The writer is very much pleased to have heard from the various members who have discussed his paper, and to have had other views than his own expressed in this matter.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

STATE AND NATIONAL WATER LAWS, WITH DETAILED STATEMENT OF THE OREGON SYSTEM OF WATER TITLES.

Discussion.*

By Messrs. H. T. Cory, O. C. Merrill, William G. Davies, W. B. Freeman, and John H. Lewis.†

H. T. Cory, M. Am. Soc. C. E. (by letter).—This paper is much more than timely, and is exceedingly interesting, because, in the Western States particularly, the water laws are of vital importance, not only to very many engineers and individual owners, but to the general public. That there could prevail, in the United States at the present time such absurd conditions as exist in California and other Western States, is impossible to grasp fully until one comes into direct contact therewith. The notable exceptions are Wyoming and Oregon; and it seems probable that within a very few years most of the really Western States, if not all, will have improved matters greatly. Two years ago the California Legislature created a State Board of Water Control and a State Conservation Commission, and one year later changed the former to the State Water Commission. At the same time, and after the passage of an amendment to the State Constitution, it made a public utility commission of the State Railroad Commission. As one result of the work and experiences of these various commissions, California will doubtless fall in line, within the next three months, with what is hoped will be the latest word in water-right legislation. Mr. Cory.

The final draft of the bill which will thus be presented to the Legislature by the administration in January, 1913, has not been made public. It is certain, however, that it will follow closely the Wyoming and Oregon precedents in general, that is, declare all water within

* Continued from January, 1913, *Proceedings*.

† Author's closure.

Mr. Cory. the State to be the property of the public; distinguish clearly between the right to water and the right to use water; make beneficial use of the basic principle—priority, purpose, period, place, and quantity; provide for determination and certification of existing vested interests, and sane supervision, control, and certification of appropriations in the future. Due to the experience of the Railroad Commission, the provisions for reviewing the acts of that body, as well as numerous other features of the Public Utilities Bill, will no doubt be adopted almost *verbatim* for governing the work of the Water Commission.

The more important of these are as follows: A commission of five appointed by the Governor, and not elected or constituted *ex-officio;* five-year appointments, with provision for recall by two-thirds vote of the members of each branch of the Legislature; provision for investigation or hearing by any one commissioner and making every finding or decision of such single commissioner, when approved by the Commission, binding; power to determine existing rights and limit them to reasonable needs; power to order improvements for preventing water waste; power to subpœna and compel attendance of witnesses and take depositions; provision that all official documents and orders filed, when certified by a commissioner or the secretary under seal, shall be evidence in like manner as originals, and may be recorded; provision that no cause of action shall accrue in any Court unless application shall have been made to the Commission for a re-hearing, prior to the effective date of orders or decision; orders of Commission must be obeyed pending re-hearing, except as Commission may direct by order; power to modify orders after re-hearing; and, finally, that only the State Supreme Court may review any finding, time of appeal limited to thirty days after rendering of decision on re-hearing, but no new or additional evidence may be introduced in the Supreme Court, and the cause must be heard on the record of the Commission as certified by it. The review is limited to determination of whether the Commission has regularly pursued its authority, including a determination of whether the order or decision under review violates any constitutional right of the appellant—in other words, the findings and conclusions of the Commission on questions of fact are final and not subject to review, such questions of fact including ultimate facts.

Further, in the Railroad Commission Act, in the absence of a definite order by the Supreme Court, the Commission's order or decision is operative, pending final decision; and, if the Court stays or suspends the operation of all or any part of the Commission's order, such stay or suspension must contain a specific finding, based on evidence submitted to the Court and identified by reference thereto, that great or irreparable damage would otherwise result to the petitioner, and specify the nature of such damage, and require filing of a suspending bond before the Court's order becomes effective.

Lastly, it is provided that all actions and proceedings under that act shall be heard and determined in preference to all civil business, except election causes, irrespective of position on the calendar. Mr. Cory.

The Railroad Commission has, and the Water Commission doubtless will have, the right and the duty to intervene in any civil action wherein the interests of the Commonwealth may, in their opinion, be concerned, and also initiate proceedings without being petitioned so to do.

It is provided that if any portion of the act be held to be unconstitutional for any reason, the validity of the remaining portions shall not be affected; and it is stated that:

"The Legislature hereby declares that it would have passed this act, and each section, subsection, sentence, clause, and phrase thereof, irrespective of the fact that any one or more sections, subsections, sentences, clauses, or phrases be declared unconstitutional."

The provision for making the Water Commission's decisions or orders binding, with appeal permitted only to the Supreme Court on exclusively legal features, is, in the writer's opinion, a very fundamental and far-reaching improvement over the procedure in Oregon or Wyoming. Several of the other features mentioned are distinct improvements, which though relatively minor, are nevertheless important.

The author's suggestion of equally stringent and positive National water laws and control, rather naturally, though quite unnecessarily, leads him to take up and discuss the water-power policy of the United States, which is practically synonymous with conservation, as politically understood. It seems to the writer that a National Water Commission, related to State Water Commissions in almost identically the same way as the Interstate Commerce Commission (in transportation matters) is related to the State Railroad Commissions, could be arranged, and fill the need exactly. General objections to "Bureaus in Washington" certainly do not apply to the Interstate Commerce Commission, and would have no weight with any other National commission clothed with powers like the California or Oregon Water Commissions or many, and especially the California, Public Utility Commissions. On the other hand, public sentiment, very effective control and regulation of public utilities, and conservation of the natural resources for the public by State authorities—especially in the "Progressive West"—have developed to such a point that it is beginning to be safe for the National authorities to begin to release their powers gradually to those of the individual States.

The essential thing is "Conservation", and whenever it can certainly be obtained by State authorities—and not an instant before—the National Government should relieve itself of the responsibility, because more flexibility and better adaptation of control to local conditions must result. The time is not yet ripe, and never may be, with

Mr. Cory. water-power and forests, for example, but it is with some other things; and the civic awakening of the American public in very recent years is one of the most striking and significant phenomena of the present time.

The author's suggestion as to the National Government developing power and supplying it at cost is exceedingly interesting, particularly because it was made at all. By virtue of their relationships, the members of this Society are probably—next to bankers and a few other types—the most politically conservative people in the Commonwealth. That a member should, in a paper before this Society, frankly and nonchalantly advocate, not only public but National ownership and operation of such a public utility as electric power, seems to the writer most significant of the rapid changes in progress throughout American society.

The writer feels strongly that such a thing would be unwise, and believes, with Mr. Dillman, that effective regulation, with a reasonable premium on able management, is preferable to public ownership, and would result in lower cost to the consumer; but, while that may be true in general, it is not always the case, and irrigation and the Reclamation Service is at least one notable exception. In opposing an idea nothing is truly gained by refusing to acknowledge antagonistic facts and arguments—usually, the effect is exactly the opposite.

The writer believes the conditions affecting the wisdom of Governmental construction and operation of power and irrigation projects in the West are fundamentally different. Power installations are almost invariably put in by well-managed corporations, and the markets are in large part now supplied by private aggregations of capital, and the effect on these by the Government supplying additional energy at cost, would be serious and really unjust. The U. S. Reclamation Service, on the other hand, always deals with projects which are entirely self-contained and so does not affect the value of other privately or mutually controlled works, even if in the immediate vicinity. Furthermore, due to chaotic and most unwise laws, no well-managed private capital would deal with the irrigation situations comprehensively or wisely. Private irrigation projects have with very rare exceptions fared so disastrously throughout almost the entire West that it is almost impossible to secure capital for such work. Indeed, that is one of the chief reasons for improved State and National water-right laws discussed in this paper.

The Reclamation Service has since its beginning expended about \$75 000 000 in irrigation construction, and is now working at the rate of about \$1 000 000 monthly. During the past season (1912) it was ready to supply water for about 1 200 000 acres out of 3 000 000 acres for which the works are being constructed. The cost per acre ranges from \$22 to, in one instance, \$95, with an average of between \$50 and \$60. In few, if any, instances has any portion of such cost been paid

as yet by the land owners, yet the selling price of land under the various projects—with the Government charges yet to be paid—has invariably increased, in some instances by more than $150 per acre. The works installed have uniformly been strictly first class, often monumental, sometimes possibly uneconomically so. In any event, the course of Western irrigation—flimsy construction—has been avoided. Mr. Cory.

In short, the Reclamation Service has been doing a grand work, and if its operations were a measure of what might be expected of Government ownership as a whole, the proponents of such a policy would have a very strong, if not convincing, case. Its publicity department has been very efficient, but, on the other hand, it has gotten its result at a marvelously small outlay, and, even so, the people, for whom the primary purpose is supplying homes, are coming to the various projects all too slowly. This is almost entirely because the "back to the land" cry is a thing in which many join but few practice. The majority of the people, like the writer, prefer to live in the city.

The Reclamation Service has made not a few mistakes—it would be miraculous if it had not, considering the magnitude of its operations—and even many of its supporters feel that a frank admission of them would have made its position, among those whose opinions are worth while, stronger than it is. Indeed, the writer has urged that comprehensive and detailed papers, with complete cost data, be presented to this Society on each of the projects, preferably all at one time, to make a volume of *Transactions*, or a series of volumes, similar to those describing the new Pennsylvania Terminal in New York City. Opportunity to discuss matters would relieve the minds and hearts of those who feel that criticism is due, and, besides, the results of engineering experience of a Government department should be published in detail for the benefit of the Profession generally, and not kept within the confines of that bureau's staff. It is earnestly hoped that this suggestion will be followed.

Nevertheless, based on a fairly complete knowledge of only the Salt River, Yuma, and Orland Projects of the Service, and the observation and study of data examined during eight years with the Harriman Lines in California, Arizona, and Mexico—six years as a maintenance and operation official, with unusual opportunities to observe—the writer is convinced that, in these three projects at least, the Reclamation Service gets more actual work for a dollar than do the Harriman Lines. Such a statement, of course, is a judgment reached in a general way, because the character of the work done by the two organizations is so fundamentally different as not to permit of analytical detailed comparison item by item. Nevertheless, the writer feels very positive in this judgment and could cite very many instances, some minor and some fundamental, in support, though naturally many of the data at hand bearing on the matter could not properly be given out. Such a

Mr. Cory. result is due to the fact that National politics has been eliminated from the Service, but still affects the Harriman Lines somewhat; that the Director of the Service has more authority and freedom in using his individual judgment than even Mr. Lovett of the railroads has, let alone the local presidents; that the head officials, and consequently the general ideas of management of these railroads, have been changed, and the Director of the Service has not; that there is much less internal politics in the Service; and the Service organization and work, as yet at least, has not brought about the sharply drawn lines of cleavage between various departments and the leveling routine of "common standard" methods, channels of communication, limitations, prerogatives, etc., of the railroad.

The writer firmly believes that effective regulation of public utilities is greatly to be preferred to Government ownership in general, and one important reason is the inefficiency of unwieldly organizations. The Reclamation Service is one of the smallest departments of the Government, and its efficiency is less than if it were smaller. A Government power service would be considerably larger, to develop any considerable percentage of unutilized water-powers, and immensely larger than any private corporation would ever be.

Mr. Merrill. O. C. MERRILL, M. AM. SOC. C. E. (by letter).—There are certain fundamental questions in connection with water rights, the determination of which is necessary before a definite and consistent policy can be adopted or executed. Some of these questions have been brought to an apparently satisfactory solution in one or more of the Western States, and a more or less standard practice has arisen concerning them. Most of the so-called "arid" and "semi-arid" States, following the early lead of Wyoming, have provided means for the determination and adjudication of old water rights, for acquiring new rights, and for distributing the decreed water among the users. The most notable exception in this respect is California. One need not go far to find the explanation. Except for certain recent attempts, of somewhat doubtful value, to alter the statutory law of appropriation, California is still struggling along under a statute and a practice devised for and applicable to only pioneer conditions. Along with this has gone a line of decisions by the Courts constantly tending to saddle more firmly upon the State the outworn and inapplicable doctrine of riparian rights.

It is true that the strict common law rule has been modified to the extent of recognizing the co-relative rights of riparian proprietors to the use of the waters of a stream, even to depletion. The Courts have also recognized rights acquired by direct appropriation, if at the time of the appropriation the lands below the point of diversion and riparian to the stream were in public ownership, a condition that at the present time is so nearly non-existent as to be negligible. The

theory has been that at one time both lands and water rights were in public ownership; that the Government provided a procedure for the appropriation of the lands, and acquiesced in the appropriation of the water apart from the lands; and that, whenever riparian lands were appropriated under the public land laws, the full riparian right to the water immediately and automatically attached to the land, subject only to two limitations: first, the superior rights of prior appropriations of water; and, second, the co-relative rights of all other riparian proprietors. Priority of right to the use of water was made to depend on priority of time of appropriation, whether such appropriation was of water directly, or of land carrying with it a riparian right. Mr. Merrill.

The only fully recognized water right by appropriation is one from a stream on which there are no private riparian lands at the time of the appropriation. When once a piece of riparian land has passed into private ownership, thereafter no right to divert water above such riparian land for use on non-riparian land can be acquired, except upon the acquiescence or neglect of the riparian owner—that is, by right of prescription. No right to divert water below riparian land can be secure against the possibility of complete depletion of the stream by upper riparian proprietors. This is the "California doctrine of appropriation." Moreover, even this is complicated by the existence of water rights in connection with the old Spanish land grants—rights derived neither by statutory appropriation nor under the common law, but in accordance with the civil law of the Spanish colonists.

The situation that exists is not one of mere academic interest; it has very practical consequences. Relying on the rulings of the Courts, certain landed interests in the San Joaquin Valley, for example, appear to have made a business of levying tribute on all appropriators of water from streams tributary to the San Joaquin River, regardless of whether the intended use had any effect at all on the amount of water which would reach the riparian lands. The writer has been informed of instances in which hundreds of thousands of dollars were paid to secure freedom from attack in the Courts, although the water was merely taken out of the stream, carried a short distance in cement-lined conduits and then returned to the stream. If the diversion had any effect upon the flow, it increased it by reducing evaporation and seepage losses between the points of intake and discharge. Even though the purpose of the appropriation be one that will not use up the water, as power development, a mere appropriator has so little standing before the Courts that it is deemed wiser to pay the tribute before litigation than to risk the payment of a higher tribute plus costs of suit after litigation. Riparian owners could hardly have been put into a better position had they been granted letters of marque against all appropriators.

Bad as the present situation is, the outlook for the future is still

Mr. Merrill. worse, unless the practice is changed. The prosperity of the State depends on its agricultural development. This will be accomplished in large part by breaking up the old Spanish grants and other large land holdings—which have hitherto been devoted to stock grazing or grain growing—into small farms under intensive cultivation. The riparian right attaches to only such lands as border on a stream through a single continuous ownership. Riparian owners may prevent by injunction the diversion of waters to non-riparian lands. The breaking up of the large holdings means, therefore, the severance of large areas of land from any water right at all, and the concentration of water rights through riparian ownership into those lands, constantly decreasing in area, which border immediately on the streams—a menacing situation if left uncontrolled.

The writer would like to be able to agree with Mr. Lewis that the case of Kansas *vs.* Colorado settled the question of riparian rights or established a precedent that the States will follow, but, in view of the decision of the Supreme Court of California on January 2d, 1909, in the case of Miller and Lux *vs.* Madera Canal and Irrigation Company (99 Pac., 502), he is constrained to believe that, as far as his State is concerned, the probability of relief from present conditions, or of being able to work out a rational solution of the water right problem, is becoming constantly more remote. The Court in this case said:

"The doctrine that a riparian owner is limited to a reasonable use of the water applies only as between different riparian proprietors. As against an appropriator who seeks to divert water to non-riparian land, the riparian owner is entitled to restrain any diversion which will deprive him of the customary flow of water which is or may be beneficial to his land. He is not limited by any measure of reasonableness."

It might very well be advisable to grant a certain amount of priority in time to the owners of riparian land, as such land, from its natural position, is likely to be more readily irrigable; but it is a violation of the common equities and of good public policy to allow riparian land owners to put to uneconomical use or to waste completely for an indefinite period one of a State's greatest resources, on the plea that sometime, somehow, they may find use for it. The riparian user of water should be restricted to precisely the same degree and measure of beneficial use as any other user. He should be given a certain time, under a statute of limitations, within which to make beneficial application of water to his land. After the expiration of that time, he should have only the standing of an appropriator in acquiring any additional rights to the use of water.

What is true of California is true to a greater or less degree of a considerable group of Western States, the so-called "semi-arid" States. In Washington, the Dakotas, Kansas, Nebraska, and probably

in Montana and Oklahoma, the California doctrine of modified riparian rights has received the sanction of the Courts. Except in those apparently rare instances where appropriations were made when the banks of the entire stream were in public ownership, the only secure right other than a riparian right is one obtained by prescription—by open and notorious adverse user against the rights of the riparian claimant and without objection from him for the period prescribed by the statute of limitations. In any event, this is concluded to be the only form of secure title that a non-riparian owner can acquire in California, regardless of the fact that the State statutes are silent in respect to any other right than that acquired by appropriation. Mr. Merrill.

Until this fundamental question concerning the acquisition of rights to the use of water is settled, either completely for or completely against riparian ownership, or in such a way as to make in all cases, riparian or otherwise, beneficial use within a reasonable time the basis of right to the use of water; unless the "dog-in-the-manger" attitude which the doctrine of riparian rights, modified or unmodified, upholds, is definitely and completely repudiated, no hope of building up an enduring policy with respect to the public control of water resources, or of securing a complete utilization of them, or of giving definite title to them, can prove anything but illusory. What possibility is there, for example, of making any final adjudication of water rights on a stream where some users claim by appropriation, others by prescription, and others by virtue of the riparian situation of their lands? The quantity of riparian lands is constantly changing. Each riparian owner has as good a right as every other. An adjudication of such rights made to-day is inapplicable to-morrow if any of the conditions as to the volume of stream flow or amount of riparian land has changed. No effective policy could be established concerning the acquisition of new rights, for such rights would be automatically acquired in certain cases by the acquisition of title to the land, in other cases they would be obtained by prescription due solely to the negligence or lack of interest of lower riparian owners, and in other cases by purchase or condemnation. Real appropriation, the securing from the State of a clear title to the use of a definite quantity of water, cannot exist in such a situation. Similarly, but little force can attach to any scheme for an administrative control of the distribution of water. No user, save he who has acquired a prescriptive right, has any claim to the use of a definite quantity of water, or a quantity sufficient for a definite area. Everything depends on the co-relative rights of the other riparian owners under the conditions of the moment. In a country dependent on irrigation, the doctrine of riparian rights leads to nothing but unending confusion. If the Courts of Oregon have allowed that State to join the roll of those which have repudiated such a doctrine, then Oregon is to be congratulated.

Mr. Merrill. Even if this first problem, the right of use, should be solved satisfactorily there will still remain certain other fundamental issues, some of which have been met in several States, but others of which, as far as the writer can learn, have not been met in any State. Without attempting to name the States which have taken action or the means which have been used in the solution of certain of these problems, the writer will merely outline what seems to him to be the fundamental questions involved in the administration of water rights in those States which have not deprived themselves of the power to name the conditions on which water rights may be acquired and retained.

Before a thorough-going public policy can be developed for the administration of public waters, it is first necessary to determine what is the underlying purpose to be accomplished, and then to provide the means for its accomplishment. The underlying purpose should be to encourage the settlement of lands, the development of agriculture, and the building of homes and communities—the promotion of the general welfare of the citizens as a whole, rather than the special profit of certain individuals. To accomplish this purpose, it is necessary to consider carefully what character of right should be granted under varying circumstances, where or in whom the ultimate ownership of such right should vest, and what limitations, if any, should be made, and what conditions imposed upon the grant.

The purpose expressed above, however often lost sight of, has actuated the disposition of the agricultural lands of the public domain. These lands have been given to those who would cultivate them and who desired to make homes on them. It has been deemed contrary to a wise public policy to allow the title to large bodies of public land to pass to one owner. The limit has been placed at that amount which an individual owner could reasonably be expected to improve and cultivate. To have given more would have resulted in withholding the excess land from use or, if the patentee sold it, in increasing the cost to the purchaser and user over what the latter might have acquired it for first hand had the excess area been left open to entry. In securing patent to public land, the individual must act for himself. He cannot obtain title as a member of an association or of a corporation. Although details differ, the same general practice of encouraging individual enterprises and of preventing speculation and non-use has also been applied to the disposition of public mineral and timber lands. Recently, however, it has come to be realized that, although this principle applies to the disposition of agricultural lands and certain kinds of mineral lands, there are other kinds of mineral lands to develop which with reasonable economy is far beyond the reach of ordinary individual enterprise, and that a new public policy and a new method of disposition must be provided for them.

In granting title to the use of public waters to the extent of

beneficial use, the practice of water appropriations has followed, in general outline, the practice in respect to the appropriations of agricultural land; but in making little or no distinction between the appropriator and the actual user, the general practice has been quite different. To accomplish the purpose which was expressed above, the title to water rights should vest in the ultimate user. The character of right which the appropriator should be allowed to acquire should depend on whether the right applied for is to be used by the applicant himself for his own purpose, on his own property, and only indirectly for profit, or whether it is to be used by the applicant directly for profit in the delivery and sale of water or in supplying the public with some commodity or service which will be derived from or be dependent on the use of the water appropriated. Mr. Merrill.

When water is appropriated by individuals, by mutual associations, or by municipal corporations for irrigation purposes on their own lands, where any profit which may be derived from the undertaking will be indirect through the products of the soil, or where these same classes of users have appropriated water for their own domestic purposes, the character of the right granted to the actual users should be that of ownership limited by two conditions: (1) that the water shall be applied to a beneficial use without unnecessary waste; and (2) that the water shall be appurtenant to the land or the locality for which it is appropriated.

The principle underlying the disposition of public agricultural lands will apply also to the use of water for manufacturing purposes or for power, if such use is confined to localized industries and to power developments comparable in extent with the agricultural operations of the homesteader; but, to give title of ownership to the use of water for manufacturing or for power in operations of the magnitude which customarily obtain, would be at variance with the general principle which has been recognized in the disposition of the public lands. Such uses should rather be treated in accordance with the special policy proposed for disposition of extensive areas of mineral lands. The title which the appropriator may acquire for power or manufacturing purposes, in any event for such of them as comprehend quasi-public uses, should be something less than ownership. Some such lesser title should also be granted the appropriator for irrigation or for domestic use when the appropriator is not the user but is engaged in a business which takes on a quasi-public character.

The primary object in the administration of water rights for irrigation, namely, to encourage the settlement of lands, the development of agriculture, and the building of homes and communities, will best be promoted if the water rights are in all instances made appurtenant to the land on which the water is used. Whether the appropriation is made by a mutual association, by an irrigation district, or by a per-

Mr. Merrill. son or corporation engaged in the business of delivering water to the land, the water right should be an appurtenance of the land irrigated, and the actual owners of the water right should be the land owners themselves. Appropriators of water for sale should not be given a water right equivalent to ownership, but only a right of agency—a license to act in behalf of those who are the real owners until such time as they may desire to assume complete control.

Similar considerations hold with respect to the appropriation of water for domestic uses. Such water rights also should be appurtenant to the lands or the localities served, such as the homestead of the individual appropriator, the district embraced by a mutual association or other organization, or to the territory included within the boundaries of a municipal corporation; and the actual owners of the right should be the users themselves. It is manifest that, in general, the right cannot be made appurtenant to specific parcels of land or other property, and that the ownership cannot be individual, as with irrigation water rights. Either the individuals must act through organizations, such as municipal corporations, or must receive the service through the agency of persons or corporations who will make the appropriations and perform the service. If the second alternative is adopted, the appropriators should be allowed to acquire from the State only the necessary right of agency—a mere license to act in behalf of the users until such times as the users may desire to act for themselves—while the real ownership should remain in perpetuity in the community served.

From the point of view of a public administrative policy, one of the most important uses of water is for the development of power. In the form of electric energy, this use extends over wider areas, reaches a larger population, and finds a more diversified application than any other use of water. Moreover, on account of the magnitude of the undertaking and the character of the services performed, organizations which handle it are almost invariably corporate, and the business itself, with few and minor exceptions, assumes a quasi-public and monopolistic character. Clearly, this use of water cannot, in general, be made appurtenant to any specific parcels of land or to any definite locality, because the use may go beyond even the boundaries of the State. Nevertheless, the appropriator, because of the quasi-public character of his business, acts only as the agent of the public which he serves. This agency should be recognized in the grant, and the appropriator should be given only a license or lease, leaving the real ownership in that widest public which is represented by the State itself.

With a complete title, or even one limited to beneficial use, the appropriator will invariably capitalize the right to its full commercial value, if no more, regardless of whether he paid for the right or

received it gratis. It is not likely that any Court will refuse to allow full value for a water right of such character in any proceeding respecting the valuation of the properties of the appropriator. If so valued, then the appropriator would be allowed to earn an income thereon, and that portion of the public served would be called on to pay it in the form of rates. Furthermore, if it should become desirable for the public to purchase the properties of such appropriator, it would be necessary to pay, in addition to the value of all other properties, the full market value of the water right which the public once owned but gave away. Mr. Merrill.

Every person or corporation engaged in a business of this character should be allowed to earn a fair, even liberal, return on the capital actually and legitimately invested, but no such person or corporation should be allowed to acquire property gratis from the public and afterward charge rates and earn an income on it. Such a situation can be avoided in part by retaining title to water rights and by giving the appropriator only a license or lease. It can be avoided wholly only by reserving in addition the right to cancel such license at any time by paying full value for all the physical property of the licensee. If the purchase is made before the licensee has had a reasonable time within which to realize the expected profit from his undertaking, a bonus in the form of a certain percentage of the value of the physical property, varying inversely as the time which the license has run, should, in the writer's opinion, be also paid. It is believed that the investment of the licensee would be fully protected if, in the place of intangible values, a bonus of three-fourths of 1% of the appraised valuation of the real property should be paid for each year less than fifty years that the license has been in force. Such a license would be in effect an indeterminate franchise—the form which best protects the interests both of the public and of the investor.

If water-right applicants are given licenses for fixed terms without the reserved right of purchase, all such licenses or grants will, as any other unexpired franchise, have a value in proportion to the time that the license has to run, unless the license is granted on the express condition that no value shall at any time be assigned to or claimed for it, or the rights granted under it, in respect to any valuation of the properties of the licensee for the purpose either of rate making or of sale.

The appropriators of water for distribution and sale, whether for irrigation or for domestic use, and the appropriators of water for power purposes are on a common footing. Each will be engaged for profit in a business which is quasi-public; each will use in his business partly his own property and partly the property of the public; each, in his use of the public property, should be considered as the agent

Mr. Merrill. of the public. This relation between the quasi-public appropriator and the public can be fully maintained only by:

(1) Making water rights for irrigation purposes appurtenant to the lands to be irrigated;
(2) Making water rights for domestic purposes appurtenant to the lands or communities served;
(3) Retaining in general public ownership water rights for power purposes;
(4) Granting licenses terminable by purchase to quasi-public appropriators of water, whether the use be for power purposes or for purposes of sale and delivery of water to others for domestic supply or for irrigation;
(5) Granting such licenses only upon condition that they shall not be capitalized for any purpose.

To accomplish the objects and to carry out the principles named above has been the chief purpose of the Federal Government in its administration of water-power sites on the public lands. The use of the public land for water-power development, both within and without the National forests, is open to all applicants who can give any assurance of a purpose actually to develop the sites. Mere speculation in power sites has been discouraged, and properly so. Only one who has been personally in touch with the situation can realize the extent of pure speculation in water-power rights. Such speculation is no assistance toward actual devolpment; it is rather a hindrance. No sooner is it learned that a municipality or an operating company is planning to make developments, than water-right locators and right-of-way applicants immediately appear, trying to forestall the municipality or corporation and hold them up for a price. The Oro Electric Corporation, of California, has recently been working up plans for additional generating stations to be located on streams within the Plumas National Forest. Within a few days after the facts became known, applications for the use of the site were filed in the District Office of the Forest Service at San Francisco for the sole purpose of securing a prior right in order to demand from the corporation a price for withdrawing. The attempts were not successful. This, however, illustrates what has come to be an established industry in some of the Western States. It is much more successful with relation to water rights than to rights-of-way across the public lands. It is the duty of any administration, State or National, to check such practices. Whatever price the speculator may secure is so much more to be added to the capital of the operating company, and the public pays interest on it forever. Just why this indirect method of getting money out of the public pocket finds so many ardent defenders it is hard to say. The ultimate effect is no different than if it were taken from the public

treasury. One method adds to direct, the other to indirect, taxes; but in either case the public pays the bill. Mr. Merrill.

WILLIAM G. DAVIES, ASSOC. M. AM. SOC. C. E. (by letter).—It is particularly pleasing to note the general optimism in regard to public rights, and especially Mr. Johnston's view that: Mr. Davies.

"Because the States have been slow in asserting the doctrine of public rights in property which is essentially of a public character, we need not fear that the public has lost title in any way."

We are impatient at the lagging of water legislation so far behind physical development and need. At times it does us good to curb our impatience and note conditions in other countries.

While France was a Roman province, it had the Roman water laws. Under the Merovingian kings, property was held under a different tenure, and all rights of ownership in watercourses and waters were vested in the rulers themselves. In feudal times the watercourses came largely under the feudal lords, but control was constantly disputed by the king.

Upon the decline of feudalism and the beginning of modern history, about 1500, Roman law was largely restored. Modern history in France began then with navigable and raftable streams as public domain, and smaller streams entirely under the control and virtual ownership of riparian owners.

From this position, of almost complete ownership and control by riparian owners, the centuries have seen them gradually lose control. They no longer have much, if any, ownership of the stream-bed, until dry; they have largely lost their rights to the water, no longer having the right of ownership, this being now the possession of the nation, but having only a right to use the water, which right cannot be forfeited by non-use.

In the United States—California for example:

True riparian rights doctrine—the right of a riparian owner to have the waters of a stream pass his property "undefiled in quality and undiminished in quantity"—has fallen. We now have the modified riparian rights; and we well know, from the history of France, Italy, and Spain, and from the strong present tendency to minimize individual rights and magnify public rights, that this modification will be a continued process, and we will end with riparian rights being nothing at all, or, at most, but a shadow of their former selves. Even now, in California, leading attorneys declare the best water right to be one acquired by prescription—adverse use.

Mr. A. E. Chandler points out* that, based on the Court decision of Anaheim Union Water Company *vs.* Fuller (150 Cal., 331), riparian rights attach to only the least parcel of land abutting on a stream,

* "Elements of Western Water Law," published December, 1912.

Mr. Davies. and that, if an owner of a tract of riparian land subdivides and sells part of his land, those portions not contiguous to the stream lose their riparian rights unless the conveyance declares the contrary, and that the land thus severed can never again regain the riparian rights by any reconveyance.

Now we have the right of "reasonable use"; but when we have a water right based on modified riparian rights and "reasonable use", there is but little difference in net results, so far as the irrigator is concerned, between this and appropriation and reasonable use. The principal difference is that, under the Oregon administrative system, the water right is readily determinable, ownership being practically as easy to determine as land ownership, whereas, under the riparian rights system, the holder is never free from the worry and expense of a lawsuit about his title to water. Some authorities have pointed out that passing a law annulling riparian rights existing before the passage of the act is unconstitutional. Nevertheless, the passage of an act annulling riparian rights not asserted at the time of passage of the act would be of use, as it would influence Courts in the constant minimizing of riparian rights.

This process took centuries in France, and it is not yet complete. Are we doomed to spend an equal time? No; for, besides the fact that the world moves faster now, we have rapid hydro-electric development as a stimulant for rapid legal development.

When a water right is for an individual irrigator or for water-power for a single grist mill, it is of much less general public importance than when it is for development of power for a public utility electric power company with a network of lines covering some great district. The economic interest of the public in a river is vastly increased by hydro-electric plants of large public utility companies. This being so, it will inevitably and necessarily follow that public rights on that river will increase and individual rights adverse to the general public good will decrease; and this is as it should be. On every hand we see the disposition to enlarge or define more clearly and defend public and community rights and minimize individual rights.

Mr. Justice Cooley, in defining a public use, as the term is applied in the law of eminent domain, said:

"The reason of the case and the settled practice of every government must be our guide in determining what is or is not to be regarded as a public use, and it can only be considered such where the government is supplying its own needs or is furnishing facilities for its citizens in regard to those matters of public necessity, convenience or welfare which, on account of their peculiar character and the difficulty of making provision for them otherwise, are proper, useful and needful for the government to regulate."

From this definition a public use depends on times and conditions. A private property to-day may become a public necessity to-morrow.

A stream of interest to but a small group of irrigators to-day may speedily become of very great interest to a whole valley using electric power. Mr. Davies.

In California the electric power development has been rapid, the merging of power companies rapid, and, in consequence, a conservation commission was appointed to report to the 1913 Legislature. It has done a very great deal of work in preparing reports. Because of the arousing and focusing of attention, the chance of getting some good water laws enacted is much better than for many years past, when irrigation interests alone were at stake.

This same general condition, however, prevails throughout the nation. The Eastern States were not at all interested in irrigation, and, before hydro-electric power development began, they could get along very well with their old laws based on riparian rights doctrine. The arid States felt the pinch of need, and so kept constantly changing their water laws. The two portions of the Nation—the humid and the arid—were getting more and more dissimilar in their water laws; but, now that hydro-electric development is growing so rapidly, there once more appears a reason and a basis for fairly similar water laws. This is no doubt the thing that must be seized on when we urge that there should be essential uniformity of water laws among the States and abrogation of riparian rights.

There is an instance of such rapid economic change and consequent rapid legal growth in Switzerland. In that country each of the twenty-five cantons had always exercised the fullest authority over unnavigable streams within its borders. This worked very well until the development of water power on a large scale began. Then the necessity for change of laws became apparent to many. In 1891 the Swiss Society "Frie Land" issued a petition demanding the insertion of the following clause in the Federal Constitution:

"All the water powers of Switzerland not yet utilized are the property of the Confederation. A federal law shall regulate all that concerns the application of this resource and the distribution of the net benefit produced by it."

In 1895, after a thorough investigation, the Federal Assembly rejected the proposition and approved the following resolutions, which sound amusingly familiar in America:

"1. The great majority of Cantons which possess and exercise the right of sovereignty over their watercourses and which often derive direct income therefrom are not disposed to renounce that right of sovereignty.

"2. In general, the public interest in the matter is safeguarded by the Cantons according to the importance of each case.

"3. The transfer to the Confederation of the right of sovereignty and of administration over watercourses would not afford any advantage to the country and would not present any source of new revenue, either to the Confederation or to the Cantons.

Mr. Davies. "4. The possibility of availing itself of the water powers in the future could very easily be reserved to the nation by careful legislation.

"5. In general, the Cantonal authorities are better able than the Federal Government to see to the development and utilization of water powers, both from an economic point of view and from that of public and industrial interests.

"6. The importance, sometimes limited, of the object in question and the securing of a rational use of the various falls, which is often difficult, require the co-operation of the Cantonal authorities specially qualified to protect the development of the local trades and industries of the Cantons."

In the years following 1895, hydro-electric development went on apace and public opinion kept an equal stride, so that in October, 1908, by a popular vote of almost 6 to 1, the people completely reversed the former decision and adopted the following amendment:

"The Federal Congress shall have supervision over the development of water power.

"The Federal Congress shall make provision for the disposition of water-right concessions, shall prescribe the terms thereof, and shall regulate the transmission and distribution of electrical energy so far as may be necessary to protect public interests and to provide for the proper development of such resources.

"All water rights to which the terms of the federal law do not extend shall be under the jurisdiction of the Cantons, which shall dispose of the concessions, regulate the same, and impose taxes and fees for their use, but such regulation taxes and fees shall not be so severe as to prevent or inhibit the development of water powers.

"The National Government shall regulate and dispose of concessions for powers located on inter-cantonal and national boundary streams, and shall determine the taxes and fees to be imposed thereon, after hearings have been granted to the Cantons interested, but such taxes and fees shall be collected by the Cantons.

"No power development in a stream located within the Union shall be transmitted to a foreign country without the consent of the Federal Council.

"The Provisions of the Federal law shall apply to water rights concessions already existing, except in cases specifically exempted therefrom by law."

Now, to come down to the present and to America: Public opinion in America is by no means ready for any such Federal law as that just quoted from Switzerland. The growth and change of public opinion here will not take the centuries it took in France for the modification of riparian rights, nor yet will it be so rapid as to take only thirteen years, as in Switzerland, for the great legal change noted. A few days ago Secretary of the Interior Fisher, in his Annual Report to President Taft, urged a "definite and comprehensive water-power policy for streams on the public domain and navigable streams not on the public domain." While good, it is not comprehensive enough, no doubt

intentionally so, as it does not provide for water rights for other things than water power, nor specifically for water rights on interstate streams. Mr. Davies.

The question is: What is the limit of Federal or State legislation now practicable or now probably possible?

Fourteen years ago,* it was stated that:

"An adequate remedy [for difficulties on interstate streams] through national legislation, it is to be feared, is out of the question * * * Any general legislation by the National Congress relative to water rights would mean an upheaval too great and too far-reaching to be practical. It would mean that a number of the States would have to make changes, not only in their laws, but in their constitutions, and would overturn the long line of established practice and decisions of the courts in dealing with this question.

"The waters of natural streams within the boundaries of the several States in the irrigated regions being, by virtue of constitutional provision, by the abandonment of control by the General Government, or by usage or the decisions of courts, the sole property and under exclusive control of these States, it is clear that reform of evils which exist by reason of defective legislation in these States, or by reason of lack of all legislation, can only be effected by proper legislation on the part of the legislative branch of the governments of these States. It is clear, too, that reform can not be effected unless the several States, in forming new legislation or correcting that which already exists, take into consideration the rights of the citizens of neighboring States as well as the rights of their own people."

A convention to consider uniform laws was urged. Several conventions under various auspices have been held. The U. S. Reclamation Service has worked out an outline form of law for adoption in the arid States. There has been a great growth in uniformity, and but little, if any, growth in diversity of water laws.

In the fourteen years since the foregoing opinion was written, there have been three great events: First, rapid development of hydro-electric plants, intensifying the need of better water laws and nationalizing the problem. Second, a great increase in the interest taken by the Federal Government in the use of water in both irrigation and power development, and a relative growth in the legal powers and claims of the Federal Government as compared with that of State Governments. So great has been this change of thought that in the opinion of Mr. Morris Bien, of the U. S. Reclamation Service, "It is difficult to find any possible grounds for the theory of State control, as to non-navigable streams within the State, which are not tributary to a navigable stream." Third, a great growth in public opinion to the effect that a much greater number of our difficulties of all kinds should be settled by administrative processes than by appeal to Courts.

* *Bulletin No. 70*, Bureau of Irrigation Investigation, U. S. Department of Agriculture, entitled, "Water-Rights Problems of Bear River" (an interstate stream), published under the direction of Elwood Mead, M. Am. Soc. C. E.

Mr. Davies. The writer proposes the following as a compromise measure, but one which accomplishes the object of limiting water rights on interstate streams in accordance with beneficial use, and making the water right appurtenant to land rather than personal property.

Pass a Federal law with provision as follows:

Provide for a permanent bureau for interstate streams and lakes; that this bureau shall have no jurisdiction except on interstate waters (and possibly on navigable rivers and on non-navigable streams on the public domain); that this bureau shall make surveys for each interstate stream, determining areas of land irrigated, area irrigable, size of ditches, points of diversion, volume of water used, discharge of stream, storage sites, power sites, dates of appropriation and use, etc.; that this bureau shall prepare a table describing land irrigated, giving date of priority, maximum amount of water allowed for irrigation of each tract, nature of use, allowable dates of use per annum, as does the State Engineer of Oregon on an intra-state stream, but ignoring State lines; that the division of water is to be based on beneficial use and the greatest good to the greatest number; that no lawsuit involving interstate right to use water on a stream may be begun in a Court until the bureau has made its survey and table of priorities, and then only on questions of law, not of fact.

There is undoubtedly much greater hope of passing such a law as this than one under which the Federal Government would take control of all streams.

Mr. Chandler states* that decisions on interstate waters to date have been mostly on small streams. In Bean *vs.* Morris, the U. S. Supreme Court, May 29th, 1911 (221 U. S., 485), said, concerning the waters of Sage Creek, an interstate stream:

"We believe that it always was assumed, in absence of legislation to the contrary, that the states were willing to ignore boundaries, and allowed the same rights to be acquired from outside the states that could be acquired from within. * * * The doctrine of appropriation has prevailed in these regions [Montana and Wyoming] probably from the first moment that they knew of any law and has continued since they became territory of the United States. * * * Before the State lines were drawn, of course, the principle prevailed between the lands that were destined to be thus artificially divided. * * * The only reasonable presumption is that the states, upon their incorporation, continued the system that had prevailed theretofore, and made no changes other than those necessarily implied or expressed."

In a case between riparian owners in California and appropriators in Nevada on West Carson River, the Court did not attempt to ascertain individual priorities, but attempted an equitable adjustment between those in one State as a whole and those in the other State as a whole.

* "Elements of Western Water Law."

These decisions being so fair, the question arises, "Why substitute an administrative body for a Court in dealing with interstate streams?" And the answer must be: "For the sake of efficiency." Mr. Davies.

W. B. Freeman, Assoc. M. Am. Soc. C. E. (by letter).—This valuable paper and the discussions thereon have been particularly interesting to the writer, because, for a number of years, he has been working in Wyoming, Colorado, and New Mexico, and has been intimately connected with matters pertaining to water rights and the distribution of the flow of the rivers of those States. At present there is no question of more vital public importance in the United States than the equitable distribution of the waters of interstate streams, involving a definition of the relation of the States to each other and to the Federal Government. Mr. Freeman.

The Oregon law, as outlined by Mr. Lewis, is an excellent one. The fact that it is working well is rather strong evidence of its merit; and it is gratifying to know that there is a successful code of water laws in actual use, for there have been many defects in all previous ones including that of Wyoming (which, heretofore, has been regarded as the model), and, in many States, the laws—or lack of laws—in existence have been utterly ineffective and confusing. The writer believes it to be desirable that all States—at least the irrigation States—should proceed immediately to adopt new laws, fairly uniform in character, with the Oregon law as a standard; then, as Mr. Lewis clearly demonstrates, the matter will be practically taken out of the Courts, where only litigation and confusion has been the rule, and put in the hands of a Board of Control capable of determining, classifying, and indexing every right in existence. In that way it will be possible to know the quantity of water available in each stream system, for future appropriations, and the conditions under which such water is available, without detriment to prior beneficial uses. That depends, also, of course, on the variation of stream flow from one season to the next; but, with discharge data available, an expert board is surely able to make a better division of waters than the Courts.

Oregon Laws.—In the discussion, several suggestions have been made for the improvement of the Oregon law. One, which is very good, is that the unit of measurement be defined more clearly. For instance, instead of stating that the maximum use is 1 sec-ft. for 80 acres, it would be much more definite to state that not more than 2½ or 3 acre-ft. per season can be used for each acre irrigated, and it would also be well to define the maximum flow allowed in the ditches under each right. In that way much trouble would be avoided in the division of water between storage and direct users, and there would be no question as to the extent of each right, regardless of the length of the irrigation season.

Mr. Freeman. Another writer suggests that the rights and duties of the State Engineer be defined more specifically in certain matters, such as his authority to grant additional rights on a stream until its flow and the characteristics thereof are well known from a record extending over a number of years. It is quite necessary that general rules be laid down for his guidance, but it would seem to be impracticable to make laws relating to minute details in matters of this kind (nearly as much so as to attempt the regulation of precipitation and run-off by statute), and a great deal must necessarily be left to the discretion of the State Engineer and the Board of Control. They, however, should be given special authority for non-approval of filings and applications pertaining to wildcat projects, or schemes of which the merit is doubtful.

On the whole, it seems that the Oregon law needs very little improvement, and, with slight modifications, there is no reason that it should not be as applicable in the humid as in the arid or irrigation States. In fact, many of the Eastern States have already discovered the benefits to be derived from irrigation, and, if one is to believe the reports of the Department of Agriculture, the practice may be extended to every part of the Union. The present uses of water in the Eastern States, where it is returned to the stream largely diminished in quantity, do not really differ materially from use for irrigation, and, of course, power use in both localities is identical. The difficulties in regard to the adoption of uniform laws seem to exist largely in the minds of those who have been educated to riparian doctrines and can conceive of use on no other basis.

Interstate Streams.—It is very evident that there is need for immediate and comprehensive legislation regarding the division of interstate waters. The present system, whereby the Courts, at intervals of a number of years, render decisions in specific cases, some of which are contradictory in character and not one of which has yet clearly outlined the relation of the States, is utterly without efficiency, and the result is that matters are constantly becoming more complicated as the appropriations continue to increase. Congress, which had the power to create an Interstate Commerce Commission, has certainly some power to legislate without fear of being overruled by the Courts in a matter which the Courts themselves have apparently been unable to decide in a satisfactory manner.

The case of Kansas *vs.* Colorado is the one most usually referred to in discussions on this subject, and the decision itself and the parts thereof have been given a thousand and one interpretations, many of them diametrically opposite, by the adherents of various sides of the question. The people in Colorado, for instance, are generally of the opinion—by virtue of the fact that the State Constitution makes the State the owner of all waters within its boundaries—that it is the privilege of the citizens of the State to divert every drop of water, if they

Mr. Freeman.

wish, from streams flowing into neighboring States, regardless of prior appropriations in those States; and they constantly refer to the result of this suit and a decision which they believe virtually sustains their contention. It is rather hard for one who is impartial, and has been trained in the doctrine of priority and beneficial use, to conceive that any Court of Justice should support such a theory, and there is serious doubt whether such was the intent of the Supreme Court. However, it is largely on that idea that the Colorado interests expect to win the case of Wyoming *vs.* Colorado, now before the U. S. Supreme Court.

Mr. Lewis demonstrates that the Kansas-Colorado decision practically asserted that the "doctrine of riparian rights, * * * is not the law, and, therefore, never has been the law" even of interstate waters. Mr. Knowles, in his discussion, shows just as clearly that the "principle of equal rights, which is the basis of riparian rights," was followed. The writer, however, believes that the learned judges, although following the language of riparian rights—because, from education and precedent, they knew no other—actually had in mind the doctrine of prior appropriation and beneficial use. In fact, it is difficult to imagine, from the conditions surrounding the case and the evidence introduced, how riparian issues could be considered; therefore, the arguments advanced by Mr. Lewis are perhaps well grounded. The whole matter, however, shows very plainly how unsatisfactory and confusing is the present method of administration by injunction and Court decisions, and that effective remedies are necessary.

The Government and the States.—Another issue to be decided is the relation of the various Federal bureaus to the States, in the administration and use of streams, intra-state and interstate. Several who have discussed this paper have taken advantage of the occasion to attack the Reclamation Service and its work, and Federal policies in general, relating to irrigation and the conservation of natural resources. This is hardly pertinent to the question; neither is it the writer's intention to defend the bureaus thus attacked. The Reclamation Service, regardless of its unpopularity in some quarters, has done a great work and has given a tremendous impetus to irrigation development, and the Conservation movement, which seems to be a good one, has been handicapped by insufficient Congressional legislation and because the powers and duties of the officials in charge have not been defined.

The present methods of administration of the Reclamation Service and other bureaus, and the grievances resulting, are traceable directly to the chaotic state of affairs resulting from a system based on government by injunction and the determination and adjudication of water rights by the Courts. In the absence of definite limitations of their powers, it was necessary for these bureaus to define their own, and it is safe to say that many which they have assumed are not constitutional.

Mr. Freeman. It is pointed out by one writer that the Federal Government recognizes the sovereignty of the States in matters pertaining to the use of water for irrigation purposes because the Reclamation Service files its applications for water rights in the offices of the State Engineers and in accordance with the regulations of the State where the project is located. This is hardly consistent with the fact that this Service has made filings in a number of instances for all the unappropriated waters of a stream, a right which an ordinary appropriator might find it difficult to sustain. Nor does it seem entirely fair, as has been done in the case of the Rio Grande, a stream rising in Southern Colorado and flowing into New Mexico and Texas, that the Reclamation Service, not only assumed the right, some ten years ago, to file on all the unappropriated waters of the stream and its tributaries north of Elephant Butte for the irrigation of land in Southern New Mexico, Texas, and old Mexico, but has been able to maintain it by rulings of the Secretary of the Interior, and thereby prevent further development in Southern Colorado and Northern New Mexico; whereas it is questionable whether the Reclamation project will be completed, for any considerable percentage of the ultimate acreage, within fifteen years of the time that the filings were first made. That proposition and the policy involved certainly seem to be at least as unfair and as much open to criticism as the one advanced by Colorado of the right of its citizens to use every drop of water within the State. It seems easily possible to improve on both of these rather arbitrary viewpoints or methods of procedure, to the end that justice be done to every one.

Administrative Control.—The foregoing illustration is given without prejudice, either to the rights of a State or a Federal bureau, but as an example of the results of the present system. Such a condition could hardly have existed if there had been a State and National system of administrative control, as suggested by Mr. Lewis, with the authority of the Commission of Control clearly defined; and, in the light of all the experience obtained, there seems to be no reason that such a Commission should not be able to operate expeditiously and successfully. It certainly should be more efficient in securing results than the workings of the present system would ever lead one to hope for.

The personnel would probably consist largely of technical experts in matters of this kind, and provision would no doubt be made that States involved in a particular controversy be given ample representation by their own authorities to act in conjunction with this Commission in the settlement thereof. When the number of interstate streams in the United States is considered, or even those in the irrigation States, and the number of appropriations on each stream along its course, it would seem that there will be sufficient work to occupy the attention of such a Commission for a long time.

The writer sees no more equitable principle to follow, in the adjudication of a stream from source to mouth, than the doctrine of priority and beneficial use, regardless of State lines, there being a clear definition of domestic and other preferred uses, and the fact also being remembered that there will be a certain return flow to the stream from the upper users. All this will undoubtedly require a great deal of study and expert investigation, but it seems inevitable that such adjudications must be made eventually, and the sooner it is done the more advantageous it will be. Experience does not lead to the belief that much relief can be expected if the Courts continue to adjust these matters. Mr. Freeman.

The first move would seem to be the adoption by the States of uniform laws, similar in character to those in Oregon. As soon afterward as possible there should be a complete determination and adjudication of all intra-state water rights, with their priorities, by the State Engineer and the Board of Control of each State. Such a movement in the States would no doubt accelerate the creation of a National Commission, which, when formed, would have the benefit of all data as to water rights of the various States, and this would expedite the complete adjudication of the interstate streams and the settlement of controversies relating thereto. The writer believes that engineers will agree that results of that kind are necessary and desirable, whether or not they agree on the method of procedure suggested. It seems certain that the Engineering Profession must have much to do with the final determination of these matters, and the more they are decided on their merits as technical rather than legal questions, the more satisfactorily and expeditiously can they be handled.

John H. Lewis, Assoc. M. Am. Soc. C. E. (by letter).—The foregoing discussion has greatly strengthened the writer's conviction that it is the duty of the Engineering Profession to lead in the reform of State and National water laws. There is no profession which is better qualified to judge as to what the laws relating to water ought to be, and no organization better fitted than the American Society of Civil Engineers to carry on a persistent educational campaign to secure the ultimate enactment of such laws. Much of Oregon's success in the administration of her water laws is due to the pioneer work along this line in Wyoming. However, on one or two points, in his admirable discussion of the principles of law, it is believed that Mr. Johnston, former State Engineer of Wyoming, is in error. Mr. Lewis.

He says that "The States have not properly administered the smaller streams which they own." From this it is assumed that Mr. Johnston means to say that:

"The waters of all natural streams, springs, lakes, or other collections of still water within the boundaries of the State of Wyoming are * * * the property of the State,"

Mr. Lewis. as provided in Article VIII, Section 1, of the Constitution of Wyoming. This, on its face, looks like an irrevocable grant by Congress of all waters within its borders to the State of Wyoming, and that the United States, or any adjoining State, can have no jurisdiction or control over the same.

Later, Mr. Johnston states that principles are of so much more importance than details of law that he does not feel inclined to discuss the question of State or National control.

It seems that this question is fundamental. It is true that the principles of priority and beneficial use, as a basis for water titles, should be understood and adopted by both the States and the Nation, but, until we can decide as to whether this system of water titles should be administered by the States or by the Nation, there is little use discussing details.

It is doubtful if the approval by Congress of a lengthy document containing the words above quoted will ever be construed as a grant of water to a State, as to do so would virtually give to Wyoming the control of all Government land within its borders, as this land is worthless without water. Such construction would be in conflict with Section 3 of the ordinance forming part of the Wyoming Constitution, providing that the State disclaims all right and title to the unappropriated public lands lying within its boundaries.

Few, if any other, States have such positive declarations as to State ownership of water. Whatever the law in the Eastern States, there must be some unappropriated waters in most streams subject to public control under any law that may be enacted at this time. It is important that the public ascertain and determine the location and extent of these public waters so that flood-water reservoirs may be constructed for the preservation of life and property during periods of excess run-off, and for the benefit of navigation, power, irrigation, and domestic supplies during the low-water period. To do this we must have some sort of an interstate or Federal administrative system, corresponding in character to that found desirable within the State for such work. The riparian owner cannot insist that those destructive floods continue to flow uninterrupted as in the past. The right to the use of all water in every part of the United States which has been put to beneficial use can in general be considered as vested. In some parts of the East, the riparian owner has perhaps some rights in the ordinary or low-water flow of the stream, notwithstanding the fact that he has not put the waters to beneficial use. The public is entitled to know the extent of all vested rights, of whatever nature, and to have provided by law some orderly method of acquiring rights in such unappropriated water, so that development can keep pace with modern demands. Knowing the extent of such rights, they can be pur-

chased or condemned, and their value to the owner will be increased accordingly. Mr. Lewis.

If we had not discussed the relative merits of county control and State control in Oregon, we would still be attempting to administer streams through a cumbersome system of county control. It is high time that all who are thinking along conservation lines get down to a discussion of the fundamental questions on which any constructive legislation must be built, that is, the proper jurisdiction for administrative purposes.

Mr. Knowles states that "There may be a question, also, as to whether the law has become sufficiently determined to justify the creation of an administrative commission," also, "If it be true that the law for all cases has not been determined, it would appear almost certain that any important contest between States, before an administrative board, would be appealed to the Supreme Court, until the universal law had become established, and a Commission might therefore be of no value at present."

We have practically no interstate or National law relating to the administration or distribution of streams. How, then, can the law ever become settled by judicial interpretation? In the absence of such statutory laws, the Courts, as new cases arise, must practically legislate on the point at issue in order to decide the matter. If Mr. Knowles expects to await the building up of a Federal administrative system by this slow and expensive system of Court legislation, then we may as well abandon the whole subject of constructive legislation. Many lawyers take this view of the question, as the chaotic condition of State and National laws is desired by a certain class of powerful corporations seeking the monopolistic control of water without use, and because litigation of this character extends over many years, and is therefore highly profitable to the legal profession.

Appeals from any administrative board established by Congress to regulate streams are certain to be carried to the highest Courts. The quicker such appeals are taken, the better. If the law is defective, it can be amended, and we will be on the high road to a solution of these many important problems. If it is upheld, such point will probably never be carried again to the Courts. A few such cases will settle all doubtful points, and the administrative board will proceed with its work, speedily and economically, and with great benefit, through the higher development and use of our natural resources.

With reference to the Oregon system of water titles, Judge Bean, of the United States District Court for Oregon, said, in a recent case (Silvies River case, 199 Fed., 495):

"I am impressed with the soundness of the view that a proceeding for the adjudication and determination of the rights to the use of the waters within the state, instituted and conducted as provided in the

Mr. Lewis. legislative act of 1909, is in effect a proceeding on behalf of the state through an administrative or executive board to have judicially settled in an economical and practical way, the rights of various claimants to the use of the waters of a stream or source of supply, and thus avoid the uncertainty as to water titles and the long and vexatious controversies concerning the same which have heretofore greatly retarded the material development of the state."

The statement by Mr. Dillman concerning two strong corporations on a stream in California which "shut out the small fry" by protracted litigation over water and then withdrew the suit and divided the water between themselves, is a good example of the kind of justice some States mete out to their citizens. By arguing the question of States' rights or Federal rights thus delaying action, we will soon find "the small fry" getting the same kind of justice meted out to them in the Federal Courts through litigation over interstate rights, irrespective of the fact that in some States good water codes may be in effect.

In the above-mentioned case, construing the Oregon administrative code, one of California's most powerful corporations was shut out of the Federal Court and was compelled to submit to the administrative officers of the State. The same tactics had been pursued for years on Silvies River, in Oregon, where litigation has been in progress for so many years that most of the "small fry" were about ready to sell out for little or nothing and leave the country. They now have some chance to have their cases tried without ruinous expense, and once for all time, which is the important point.

Mr. Dillman wishes to be delivered from any more Government Bureaus, but if only a Government Bureau can protect "the small fry," then by all means let us have another bureau. California, however, needs a State Water Bureau worse than any other State in the Union, and the lack of such bureau is probably due to the presence of such powerful companies having control of great quantities of water under lax laws which do not require beneficial use as the basis of the right.

The writer is glad to note the admission by Mr. Sheley that the attempt to adjudicate water rights in Utah through proceedings in the Courts has been found to be impractical. This is the history in every State where an earnest attempt thus to adjudicate water rights has been made, and is the reason why many States are now turning over all water matters to administrative boards having only subordinate judicial powers.

Mr. Knowles believes that, by applying the "rule of reason" to the two conflicting doctrines of riparian rights and appropriations and use, both can be merged into one and the conflicting laws of adjoining States be thus brought into harmony. This plan has been attempted for years in California. It has resulted in the most confused jumble

of impractical laws to be found in any State in the Union. What the law is in that State no one can tell without examining thousands of pages of conflicting Court decisions, and then no two investigators of the law can agree exactly as to what the law really is. The rule of reason applied by one judge will differ from that applied by another. In those large interstate or National problems like the control of the Mississippi or Ohio Rivers, where millions of dollars of the peoples' money must be invested, we must have some absolutely definite legislation before such work is undertaken. Such works should endure for all time, and the necessary title to water should not be left to the reason or logic of any judge, however learned. Mr. Lewis

In his discussion, Mr. Knowles has attempted to point out certain errors in law made in the paper. As Mr. Johnston says, "Principles are of so much more importance than details of law," and the writer believes it is almost a waste of time for the engineer to discuss such detailed matters. Therefore, no attempt will be made to point out what are believed to be errors in his reasoning.

We must recognize the fact that we have at the present time no interstate or National administrative laws relating to the diversion and use of streams. It should be the province of the engineer to ascertain what the law ought to be. When this has been accomplished, and such law is framed in harmony with the laws of Nature and in conformity with good engineering and business practice, then legal experts should be called in to advise what changes are necessary to conform with good legal practice. Some of the best lawyers are always found on the side of reform to promote the public good. Owing to the present confusion of water laws, it will doubtless be found that almost any law for the highest public good can be sustained in the Courts. We are certain, however, that every section of such law which conflicts in any way with vested property rights will be pointed out by the lawyers in Congress, and most fully corrected before any such legislation will be enacted. However, it is believed that Congress will welcome suggestions from the Engineering Profession as to what the law ought to be, in order to stimulate the development of vast engineering enterprises.

DIVIDED CONTROL.

Since writing the paper, the Department of the Interior, of the United States, has issued regulations concerning rights of way through the public lands and reservations of the United States * * * for Power Purposes, approved August 24th, 1912. These regulations apply to Interior Department lands, and are much the same as those adopted by the Agricultural Department for lands within the National Forests. They are based on the theory of land control, impose certain limitations on the use of water, and amount to the same thing as the United States controlling the water. Permits are revocable at will by

Mr. Lewis. the Secretary, and terminate in fifty years unless renewed under certain conditions.

These regulations have confirmed the writer's opinion that the United States controls not only the public lands in the Western States, but also the water flowing over the same, and that the State exercises almost no control whatever over such waters, except by virtue of these regulations which require compliance with State laws. When such lands pass into private ownership and water is appropriated, then such water to that extent is subject to State regulation.

The Acts of Congress of 1866 and 1877 are believed by many to be an irrevocable grant of water to the States, but this, apparently, is not the case.

Concerning this question of State control of waters on Federal lands, the Supreme Court of the United States held, in the case of the United States *vs.* Rio Grande Dam and Irrigation Company (174 U. S., 690-703), that:

"Although this power of changing the common law rules as to streams within its domain undoubtedly belongs to each state, yet two limitations must be recognized: (1) That in the absence of specific authority from Congress, a state cannot by its legislation destroy the right of the United States, as the owner of lands bordering on a stream, to the continued flow of its waters; so far at least as may be necessary for the beneficial use of the Government property; (2) That it is limited by the superior power of the general Government to secure the uninterrupted navigability of all navigable streams within the limits of the United States."

"In other words, the Court holds that the jurisdiction of the United States over the natural watercourses upon the public domain" is superior and paramount to the jurisdiction of any State; and that all needed measures may be taken by the Government to preserve the watercourses of the country for "at least" the two purposes named above "even against the action of any state," in authorizing, under its laws, appropriations to be made. The Court, especially in the Kansas-Colorado and Rio Grande cases, clearly intimates, to say the least, that the Government might also make other claims to the water than for its use for irrigation, or as a riparian owner. Whether it will do so, time alone will tell.*

"The Government is still the owner of the surplus of the waters flowing upon the public domain, or rather the owner of all the waters flowing thereon, remaining after deducting the rights to the use of the same which have vested and accrued in some legal way to individuals and companies. * * * It therefore follows as the result of the ownership by the United States of the waters flowing on the public domain, that any dedication by a State of all the waters flowing within its boundaries to the State or to the public, amounts to but little, in the

* Kinney, in his recent work on Irrigation Law, p. 1096.

face of any claim which might be made by the Government, at least to all the surplus, or unused waters within the State."* Mr. Lewis.

Kinney states, further, that Congress, if it had seen fit, could have laid claim to the necessary surplus water for projects constructed under the Reclamation Act, by "virtue of the fact that title to the surplus waters flowing over the public domain is in the Government," and not under the State laws, as was provided in Section 8, of such Act. Much confusion within the States may result eventually where Congress has reserved water for Indian reservations, which, after many years, has never been used by the Indians, but, in the meantime, has been used by others below.

It appears, therefore, from these authorities and from the wording of the Acts in question, that Congress, if it saw fit, could repeal at any time existing laws relating to water and adopt some new law for all surplus and unappropriated waters on the public domain. In such an emergency, we would have divided control within the Western States to such an extent as to make any control impracticable except through the closest co-operation.

Mr. Cory states that it is beginning to be safe for the National authorities to start gradually releasing their water powers to the individual States, and that National ownership and operation of such a public utility as electric power would be unwise.

The writer will attempt to show that in some cases the first proposition is unwise, and that the second proposition is the only logical plan for the development, distribution, and use of water power in the Western States.

Transfer to States.

Owing to the conservation sentiment in the Eastern States, it is believed to be very unlikely that the United States will ever turn over to the States, for administrative purposes, the control of public lands now withdrawn for power purposes. We cannot await the settlement of this complicated political issue before undertaking some of the large enterprises in the West, which per unit cost are far cheaper to construct than many of the small projects now being undertaken by private capital, or by the public under the Reclamation Act, or Carey Act, without co-operation with all other interests in the field. To illustrate the point, one large irrigation project and one large power project in Oregon will be described, and an attempt made to show that immediate construction is what is wanted, leaving to the future the settlement of the question of State or Federal control of streams.

Deschutes Irrigation Project.

On the Deschutes River, at Benham Falls, 181 miles measured along the stream from its junction with the Columbia River at Oregon, a

* Kinney, on Irrigation, Second Edition, p. 692.

Mr. Lewis. 75-ft. dam will back the water up for more than 20 miles, flooding 25 000 acres, and will store 700 000 acre-ft. of water. The remainder of the 1 053 000 acre-ft. of water which passes this point annually can be diverted from this stream in the vicinity of the Town of Bend, without storage, for irrigation during the summer, and will supply about 100 000 acres of land. The stored water will irrigate about 210 000 acres in addition. The entire stream, at Benham Falls, will thus supply 310 000 acres, which is equivalent to a strip of land 1 mile wide and 484 miles in length. It has been roughly estimated that the cost of construction will be about $30 per acre.

Incidental to this development, an enormous amount of summer power will be produced, for, in the 37 miles between the dam site and the last diversion dam for irrigation purposes, there is a total fall of 1 300 ft. During the height of the irrigation season, about 5 000 sec-ft. of water will be available for power development in the upper portion of this stretch. As the different diversion dams for irrigation are passed, smaller quantities of water will be available for power until the last diversion at Cline Falls is reached, where about 1 500 sec-ft. will be diverted for the lower districts to be irrigated.

During the winter, when the reservoir is filling, the power possibilities along 70 miles of the stream channel, having a total fall of 2 600 ft., will be destroyed, but this is not a serious loss, as the seepage water below the reservoir can be carried through the lower irrigation canal to a vertical drop about 800 ft. in total height, where more winter power can probably be developed than can be used for many years to come in that section. Near the lower end of this district which it is proposed to irrigate, the stream enters a deep rock-walled cañon from which it is impossible to divert water. Some of the largest tributaries enter just below the section to be irrigated, so that, with full storage at Benham Falls, about 4 500 sec-ft. can be depended on at the mouth of the stream, provided 80 000 acre-ft. can be stored on Crooked River, one of the upper tributaries, to piece out the low-water flow. There is a fall of 1 400 ft. in 111 miles of this lower river, below the mouth of the Metolius River, its principal tributary. On this latter stream there is a fall of about 2 600 ft. in 70 miles. Practically all of this enormous fall can be utilized. The flow of the stream is most peculiarly uniform, due to the light porous pumice stone formation which covers the entire upper drainage of the Deschutes and its principal tributaries to a considerable depth.

These enormous power and irrigation possibilities are so interrelated that the United States could not surrender the power sites without jeopardizing its land and irrigation rights. The United States owns about one-half the irrigable land in this district, and has withdrawn enough land along the stream practically to control the power situation. So enormous are the power possibilities that ultimately

much of the Government land along the Columbia River and other streams, both in Oregon and Washington, within economical pumping range, will ultimately be reclaimed by power from the Deschutes River. Mr. Lewis.

To settle finally all the complicated State and National questions, so as to permit the logical development of this stream for the greatest good to the greatest number, would require many years. In the mean time, the railroad which is now constructed up the Deschutes River to the Town of Bend, will have been constructed southward through the proposed reservoir site where its right of way is now partly cleared. Also, numerous power plants will have been constructed along the main stream immediately below the proposed dam site, thus preventing the storing of the winter flow. These and other like complications, if not prevented, will accumulate to such an extent that in a few years this magnificent project will be too much encumbered to undertake; but all such difficulties can be prevented without in any way retarding present development, if such private development is forced to conform to some public plan for the stream as a whole.

The development of much of the water power on the Lower Deschutes River, has been greatly complicated by the recent building of a railroad along either bank of the stream so near the water surface that the construction of dams to an economical height is made impossible without moving the grades. These roads were forced to elevate their tracks around the proposed power sites of two private companies, also one site selected by the U. S. Reclamation Service. With but little, if any, extra cost, these roads could have been constructed at a uniform height of 100 ft., or more, above the bed of the stream. The water power, however, belonged to the public, and we have been so busily engaged, in disputing the right of the State or the Nation to control, that, in this instance, much power has been almost completely destroyed through public neglect. The same will occur with respect to the irrigation and power possibilities of the upper valley, where only 65 000 acres are now under irrigation, and where there is a possibility of irrigating 500 000 acres if the water is put to use in accordance with a comprehensive preconceived plan.

The State has a large measure of control over the waters of this stream, and the United States, through its National Forest Reservations, Indian Agencies, and water-power withdrawals, exercises a very large control over such stream through the control of land rights. Therefore, the public interest could be well protected if the State and the Nation could unite in some workable plan for the greatest good.

Co-operation.

On February 21st, 1913, the Legislature of the State of Oregon appropriated $50 000 for co-operation with the U. S. Reclamation Service in the preparation of a comprehensive plan for the Deschutes

Mr. Lewis. River Basin, and the Secretary of the Interior has agreed to allot a like sum for such work, which is to be carried out jointly.

The State law authorizes the withdrawal of all necessary water rights for the protection of the project. On the completion of the plans, any part or all of the project can be turned over to private capital for construction, on payment of the full cost of preparing such plans, the money to go into a revolving fund for the promotion of other projects; provided that no such project, or any part thereof, shall be turned over to private capital except on full hearing of all interested parties, and proper assurance that public interest will be safeguarded.

Anticipating difficulty in securing private capital to carry out so large an undertaking, the Legislature has recently submitted a constitutional amendment for the vote of the people, authorizing the issuance of State bonds to the extent of 2% of the assessed valuation of the State for the construction of irrigation and power projects, and for clearing and developing the cut-over timber land of the State.

In case it is found necessary for the public to undertake the development of its own resources, it is being strongly urged that the State co-operate with the Nation, rather than attempt a new organization. It is believed that the people will not vote millions of dollars for development projects without tying up to some stable, experienced organization which has actually made a success in this important work. The few mistakes of such service only serve to strengthen the arguments for co-operation, as these mistakes are not likely to be repeated. In this way the difficult question of politics and patronage can be overcome. It is believed that the United States can be prevailed on to co-operate with the State, dollar for dollar, as $20 000 000 has already been voted by Congress for irrigation in the West, without any offer of co-operation on the part of the States most vitally interested.

It thus appears that the policy most likely to secure immediate development is that of co-operation between the State and Nation, leaving to the future the discussion of States' rights and Federal rights. If we should take the position, advocated by Mr. Cory, of demanding the turning over by the United States to the States, of its various resources in the West, we might never attain that end. If we did, we would then be many years behind the times, as each State would have to organize a complicated technical bureau to handle the work, and would inevitably repeat many of the mistakes of the United States, if not do far worse.

In a very brief campaign, we have actually attained in Oregon co-operation with the United States. Furthermore, the people are in entire sympathy with a comprehensive development programme, as the Legislature has just appropriated $450 000 to complete one unit of the big Deschutes scheme, where the State has made a failure in

the reclamation of 27 000 acres from Tumalo Creek, under the provisions of the Carey Act. Water rights to about 20 000 acres of land had been sold where the supply was barely sufficient for about 3 000 acres. Mr. Lewis.

Power Development by the Public.

Each house of the Oregon Legislature has passed a somewhat different bill appropriating $25 000 for the thorough investigation of the Columbia River Power Project, near The Dalles, as outlined by the State Engineer in his Fourth Biennial Report. It is believed that an agreement will be reached within the next few days and the investigation authorized. It is the purpose of this bill to make diamond-drill borings, prepare detailed plans, specifications, and estimates of cost for a 536 000-h.p. project, and to estimate the cost of transmission of this power to various points in Oregon and Washington; also to estimate the cost of distributing such power from sub-stations in the cities to individual consumers. To show the saving which would result, estimates are to be furnished giving the cost of producing power from other sources than water. This work is to be carried out jointly by the States of Oregon and Washington, and the United States, if co-operation can be had from such sources. A joint legislative commission from both States, headed by the respective Governors, has examined the project and recommended the plan. In order to show why private capital cannot undertake the construction of this project, and why it is necessary for the public to promote it, and perhaps eventually construct it, a brief description of the entire plan is presented.

Columbia Power Project—Summary.

Installed Machine Capacity: 536 000 h.p. with 300 000 h.p. of 12-month power and 236 000 h.p. of 8-month power.

Location: At Big Eddy, 3 miles above The Dalles, 90 miles east of Portland, the metropolis of Oregon, and 200 miles from the mouth of the Columbia, and the Pacific Ocean.

Market: 240 000 h.p. for fertilizer works, the remainder for the iron and steel industry, wood distillation plants, aluminum, carborundum, carbide, alkali works, electro-chemical industries, woolen mills, pulp and paper mills, light, heat, and power in wholesale blocks to encourage new industries, and without competition in retail business of local power companies.

Dam: Present channel of Columbia to be closed by a dam, 300 ft. long, approximately 180 ft. high above foundations, and new channel excavated in solid rock 1 400 ft. wide, and water surface regulated by removable dam, sections 100 ft. long, 60 ft. high. Location, 1½ miles above power-house.

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Power-House: Oregon or Washington side. Planned for Washington side, at Big Eddy; 1 200 ft. long, about 200 ft. high above foundations, contains 21 turbines, supplied by canal, 300 ft. wide, with 20 ft. depth of excavation, $1\frac{1}{2}$ miles long from pool above dam.

Head of Water: One-half height of Niagara Falls (73 ft. at low water; approximately 42 ft. at high water). Natural fall at low water in river 11 ft., with fluctuation of 90 ft. at proposed dam site.

Water Available: Minimum, 50 000 sec-ft., maximum, 1 390 000 sec-ft., mean flow, 235 000 sec-ft. Drainage area at The Dalles, 236 800 sq. miles. Stream flow record for 33 years available.

Turbines: Runner 16 ft. in diameter, shaft of steel 30 in. in diameter, 60 ft. long, supporting generator on top approximately 36 ft. in diameter, maximum capacity, 32 000 h.p., all at 80 rev. per min. on one oil bearing. Maximum water capacity of each, 5 000 sec-ft.

Cost: $23 076 000, or $77 per h.p. for 300 000 h.p. Cost of low-tension power station, $6.90 per h.p. per annum. Cost of power by steam from waste sawdust, about 0.5 to 0.7 cents per kw-hr., or from $33 to $46 per horsepower-year. Minimum price at Niagara for large blocks, $9.00, average about $15.00.

Transportation: Competing transcontinental railways at power-house, with navigable water from same to Pacific Ocean, and for many miles inland.

Raw Products: Abundant and accessible for many different industries. Air, 20 000 000 tons of nitrogen above 1 sq. mile of the earth's surface, or enough to supply fertilizer for the world for 50 years. Timber, iron ore, lime, salt, and other products accessible because of rail and water transportation.

Market for Manufactured Products: The World; as the power plant is at the gateway to an inland empire, with water outlet to the Pacific. With public docks at Portland, and the Panama Canal completed, shipping facilities will be available for industries seeking world markets.

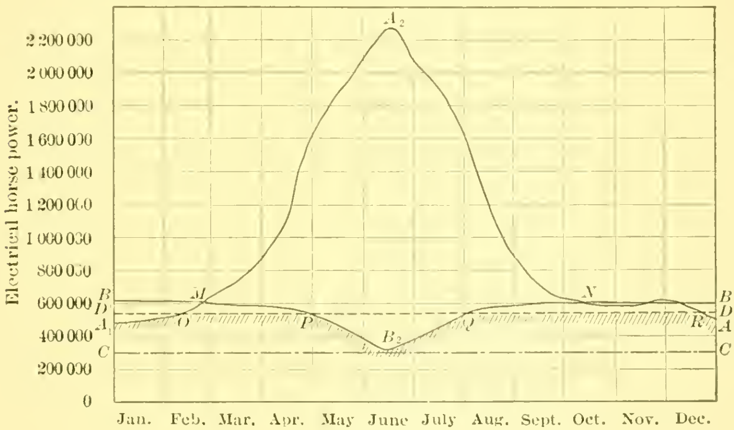
Low water in the Columbia River invariably occurs during the winter, and floods during the summer. These floods are caused by the melting of snow in the high mountains at the head of the stream.

Fig. 6 shows the total power available in the Columbia during an average year. It is based on approximately maximum head conditions, and made up from records of the last 10 years, with an assumed turbine efficiency of 80%, a generator efficiency of 93%, and on the

assumption that sufficient turbines are installed to use all the water at all stages. Mr. Lewis.

The curve, B_1 B_2 B_3, in this diagram shows the power which could be delivered to the switch-board by twenty hydraulic units, assuming the generator large enough to transform the power without overheating.

The line, *DD*, is the normal rated generator capacity, 400 000 kw., or 536 000 h.p. The lower of the three lines at any point is the maximum power output of the station. Thus it will be seen that at low-water periods the flow of the river limits the station capacity (A_1 O and R A_3); at intermediate discharge, the generator capacity limits (O P and Q R); and at high-water periods the turbine capacity is the controlling feature (P B_2 Q). The resulting minimum capacity is represented by the shaded line. The line, *CC*, is the adopted minimum station capacity of 300 000 e.h.p., or represents the line of perennial power, sometimes called "primary power," or that which is deliverable with reasonable certainty under all conditions.



POWER DIAGRAM FOR AVERAGE YEAR.

FIG. 6.

The shaded line, at nearly all places, is far above this line, *CC*, indicating that much surplus power would be available at certain seasons of the year if customers could be found whose operations could be temporarily suspended during floods and during extreme low water.

Thus it will be seen that, during practically all of February, March, April, August, September, October, November, and December, or 8 months of the average year, a total of 536 000 e.h.p., or 236 000 e.h.p. of surplus or secondary power, could have been delivered to the switch-board without any additional capital investment.

During the summer irrigation season of nearly 5 months, as much as 500 000 h.p. in addition could be developed at an initial cost of

Mr. Lewis. about $44 per h.p. and furnished at about $4.50 per horse power-season. This power could be transmitted along the river for pumping to arid lands in both Oregon and Washington. It is this power that the United States should be interested in; and this is the reason that the Reclamation Service should co-operate in the investigations. (The writer has been assisted in working out the technical details of this project by Messrs. L. F. Harza and V. H. Heineking.)

The proposed dam will flood about 6½ miles of the navigation canal which is now being constructed by the United States at a cost of several million dollars. The lower 1½ miles, containing three locks, will be utilized in connection with the proposed power project, the boats entering the stream immediately above the dam and proceeding up the wide open river. At low stages, 10 ft. of water will be found above Celilo Falls. Owing to this saving in construction cost, the ease with which boats can go up the wide river, in place of the long, narrow, shallow canal, it is believed the United States should not only co-operate in the investigation of the project, but also in its construction.

When this navigation canal was projected there was no prospective market for 536 000 h.p. To-day it is believed that the entire quantity could be disposed of within a short time after the completion of the plant, and to new industries not now located in the Northwest. The construction of this project would mean the investment of perhaps $100 000 000 of new capital in this district, besides affording profitable employment to thousands of laborers, and add greatly to the taxable wealth of the State.

This statement relative to market is based on correspondence with large power users in different parts of the world. The representative of certain foreign capital has examined all power possibilities on the Pacific Coast, including this one. After going over the proposed project, as briefly outlined above, he agreed to recommend to his company the purchase of 240 000 h.p. at $9 per horse power-year, delivered at the low-tension bus-bars of the generating station, putting up a $200 000 surety bond, provided the State would contract for 40 years, the power to be used in making fertilizer from the air.

This sale alone, amounting to $2 160 000 per annum, would fully finance the entire project for the development of 536 000 h.p., paying interest, depreciation, operating and maintenance expenses, carefully estimated at $2 068 000. It is probably true that some of the remaining power could be disposed of at from $30 to $50 per horse power-year to the railway lines in this vicinity, but only a very limited market could be found at such figures. This one plant could supply a city thirteen times the size of Portland to-day, or one of nearly 3 000 000 inhabitants. Forty years hence the population of Portland may reach this figure.

NECESSITY FOR CO-OPERATION.

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One-half the bed of the stream, and of the water, is subject to the control of either State, and the United States controls all matters relating to navigation.

Private capital would have extreme difficulty in negotiating with these three agencies, and would run considerable risk through subsequent legislation detrimental to their interests.

The Oregon Legislature recently came within a few votes of imposing a heavy annual license tax for regulation purposes on all vested water-power companies, and the Washington Legislature has now under consideration a bill providing for the forfeiture of water rights in that State if electricity from a plant now under construction is transmitted across the river for sale in Oregon.

Oregon now limits franchises to the use of water for power purposes to 40 years, and imposes an annual tax of from 25 cents to $2 per horse power-year, depending on the percentage of power appropriated which is put to beneficial use. Partly for this reason, power plants prefer to locate just over the line in Washington. The United States would probably insist on some different franchise limitation, and the payment of heavy charges to the Federal treasury for revenue or for the improvement of the stream. The State charges alone, or the annual payments to the Federal treasury, would defeat this project, as there is even yet no market for this power except at extremely low prices.

The limit for fertilizer works is, perhaps, $9 or $10; for iron and steel works, $7; for aluminum, alkali, carborundum, and carbide works, $10; and cost or less for heating purposes. Much power could be used in heating retorts for the distillation of waste wood products, in each cord of which, as stated on reliable authority, there is $18.40 in value, as follows:

4 gal. of turpentine, at, say, 30 cents	$1.20
85 lb. of calcium acetate, at 12 cents	1.70
2 gal. of alcohol, at 50 cents	1.00
40 gal. of tar	8.50
1 000 lb. of charcoal	6.00
	$18.40

The utilization of this waste wood would thus greatly stimulate the development of our already extensive lumber business, and the making of pig iron in the electric furnace would afford a market for the charcoal thus produced.

Pig iron is now being produced successfully at Heroult, Shasta County, Cal., where a plant has been in operation for the past six years. It is stated on good authority that the experimental stage of

Mr. Lewis. this industry is definitely past, and the recent enlargement of the plant appears to confirm this fact.

Electrical metallurgists, such as Heroult, are now very confident that, when the new method is installed on a large scale, it will be feasible to attain a yield of 12 kg. of pig iron per horse power-day. Under such conditions, current at $9 per horse power-year could compete with $4 coke.

At such rates there could be but little if any profit for private capital in the electrical end of the business, and few enterprises which have but recently passed the experimental stage can afford to risk $23 000 000 in a power plant in order to get a small block of cheap power. The present retail power rates in this vicinity vary from $74 to $460 per horse power-year, and thousands of horse power in the State can be developed and delivered to consumers in reasonable quantities at perhaps $15 to $25 per horse power-year. With the opening of the Panama Canal, it is believed that the development and sale of power by the public at little, if any, profit will do far more to stimulate industry and promote the peace and general prosperity and welfare of the people in the Western States than can ever be accomplished through irrigation, and that, without such public development and distribution of power, this development will be forever denied to this section of the United States.

For the same reason, therefore, that the United States was compelled to go into the irrigation business in order to dispose of its arid lands, and for the same reason that the State is now planning to co-operate with the United States in the irrigation and colonization of both public and private lands, so the State and Nation must go into the power business if its untold wealth of water-power resources is ever to be put to use. In Oregon, more than 3 000 000 h.p. is going to waste in the streams, to produce which in steam engines would consume $144 000 000 worth of coal annually. This economic waste is enormous. Why should Oregon impose heavy taxes on power development and the companies pass this on to the people, thus maintaining power prices the same as in coal-producing regions, when, by basing the price on cost of construction, power could be sold at such low rates as to give the western section a tremendous advantage over the eastern section which has enjoyed for years an unfair economic advantage over the Northwest in having cheap coal?

The capital now invested in the exploitation of the natural fertilizer deposits in Chili is $136 000 000. It would require a capital investment of $860 000 000 to assure the production of an equal quantity of artificial fertilizer from the air by means of the electric furnace, assuming that cheap power were available.* It is proposed that Oregon

* Special Agents Series No. 52, p. 76, U. S. Dept. of Commerce and Labor.

adjust her laws so as to compete with the world in attracting these new industries. Mr. Lewis.

It makes little difference whether the State or the Nation backs the bonds for irrigation works, as long as such works are built speedily and economically, and the lands are colonized so that the money is returned with interest. Likewise, it makes little difference who owns the land or the water at feasible power sites, so long as these sites are developed economically and the power put to use. Co-operation for the speedy construction of these projects should be the aim of all, leaving the control to the State or the Nation in proportion to the funds advanced. It is a waste of time to talk about State or National control at this stage. These matters will all be adjusted later.

TERRITORY AFFECTED BY COLUMBIA AND MISSISSIPPI RIVER POWER PROJECTS.

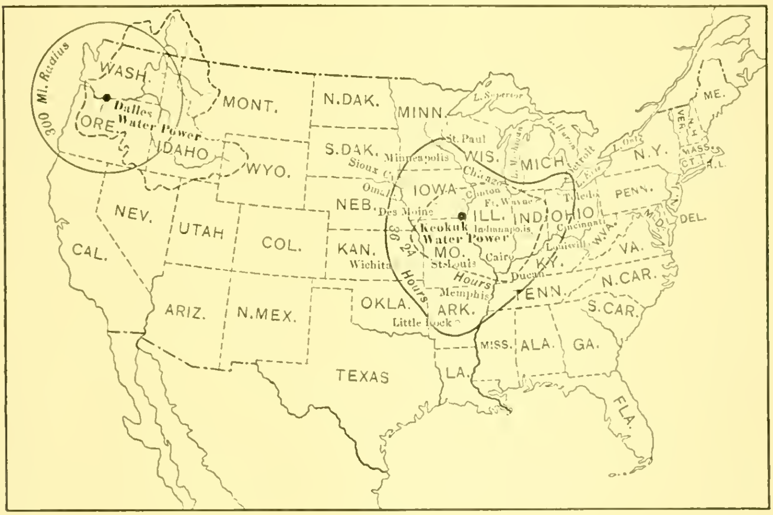

FIG. 7.

ADVANTAGES IN CO-OPERATION.

There are many advantages in co-operation. From Fig. 6 it will be noticed that approximately 2 000 000 h.p. can be developed in the Columbia near The Dalles, during June of each year. When the Columbia River is at low stage, almost all the other rivers in the Northwest are at high stage. Power from this plant could be transmitted, from 300 to 400 miles, at prevailing commercial rates, if no other cheap power were available. From Fig. 7 it will be observed that the entire Northwest falls within a radius of 300 miles from The Dalles. The demand for power in this territory varies from hour to hour, and from season to season, with as great fluctuation as stream

Mr. Lewis. flow. By having all plants in this section tied into one system, supplying all consumers, a tremendous economic advantage will result. No individual State can ever accomplish such a comprehensive scheme, nor could the United States acting alone. Any independent action, by either State or Nation, will result in endless complications, such as are already beginning to be felt in the Northwest through the divergent laws and interests of different States.

Private capital is aware of the economies attained by consolidation, but the people favoring public owership and control are many years behind the times. Each town is attempting to supply its own citizens for special uses. As a result, from two to four times the necessary machine capacity must be installed to carry peak loads, and the plant is practically idle for a large part of the time each day. The cities, perhaps, can take care of local distribution, but the States and the Nation should take care of the generation and transmission of power.

The information on Fig. 7, relative to the Mississippi plant at Keokuk, was furnished by the Keokuk Industrial Association, and is intended to show the very great territory affected by cheap power. According to this authority the 200 000 h.p. to be developed at the Keokuk plant will cost \$125 per h.p., in comparison with \$77 estimated for the Columbia River plant. The difference in cost prices at which power could be sold from these plants will, for some products, more than equal the freight rate from Portland to Chicago.

In the Eastern States, where water power is somewhat limited, and where all the people cannot enjoy equal advantages from the distribution of power at low prices, it may be found to be better policy to impose annual charges on water-power companies, so as to offset the special privilege enjoyed by such companies in competition with power produced from coal, the revenue thus secured going to reduce the general burden of taxation. On thorough consideration, the water powers in all sections of the United States may be found to be ample, so that the same policy could be adopted for the Nation as a whole, using Government coal burned at the mine to supply cheap power to those sections unfavorably located with respect to cheap water power.

The present era of unregulated competition must come to an end in the near future. In its place we will have a National water-power trust, in either private or public control. A water-power expert who is familiar with hydro-electric power conditions throughout the world recently expressed surprise at the extensive development of water power in California and the very limited market. The writer has what is believed to be reliable information to the effect that a 200 000-h.p. plant is now under construction within 200 miles of San Francisco which expects eventually to deliver power in that city at \$10 per horsepower-year, or \$8 less than can be supplied by its nearest com-

petitor. The result is inevitable. The water-power trust must purchase this plant, sooner or later, thus adding additional equipment which is not needed at the present time, and the people will have to pay dividends on such "hold-up" prices. Owing to such extravagance, which is forced on the company in control, power rates cannot be greatly reduced to stimulate sales, and the new industries requiring large quantities of cheap power cannot secure a foothold in such State. Mr. Lewis.

Mr. Cory has had wide experience in railway work. He admits the successful handling of reclamation work by the U. S. Reclamation Service, saying that the "Reclamation Service gets more actual work done for a dollar than the Harriman Lines do," in certain cases coming under his observation. He says:

"Such a result is due to the fact that National politics has been eliminated from the Service, but still affects the Harriman Lines somewhat; that the Director of the Service has more authority and freedom in using his individual judgment than even Mr. Lovett of the railroads has, let alone the local presidents; that the head officials, and consequently the general ideas of management of these railroads, have been changed, and the Director of the Service has not; that there is much less internal politics in the Service; and the Service organization and work, as yet at least, has not brought about the sharply drawn lines of cleavage between various departments and the leveling routine of 'common standard' methods, channels of communication, limitations, perogatives, etc., of the railroad."

The writer has had several years' experience on engineering work for the Harriman system, has served in the U. S. Reclamation Service for about the same length of time, and for the past eight years has had charge of water right and irrigation matters for the State of Oregon. He is thoroughly conversant with the value of system, and of thoroughness in the perfecting of a large organization for a complicated technical work. He is also aware of the instability of the average legislature, and its lack of respect for stability and permanence in organization. Often the person seeking a job has more influence with the State Legislature, through influential members, than a responsible State officer elected by the people and under bond to perform his duty. There is no opportunity in State work for a young person to enter the service as a life occupation. Merit is not, and cannot be, rewarded under such an unstable system with frequent changes. Where work is intrusted to a board, and new officers are elected every few years, a conscientious officer is likely to see years of patient work along some definite policy destroyed by the adoption of some new and antagonistic course of action which the inexperienced majority honestly believe to be the best, at least the best for personal political reasons. Salaries are never equitably adjusted.

Mr. Lewis.

State Work.

With this experience in private and public work, the writer had come to the same conclusion as Mr. Cory with respect to the success of Government irrigation. The criticisms offered by the general public are usually of a minor nature, and little is ever heard of real engineering mistakes. The present difficulty of the Service in colonizing its lands can easily be overcome through co-operation on the part of the States which are most vitally interested. This is the weakest part of the Reclamation Act.

Believing that it is preferable to expand an existing organization which has already made a success of its work, than to create a new and independent organization to work in a complicated field without harmonious relations with the larger organization, the writer is now urging co-operation between the State and Nation in the construction of both irrigation and power projects.

By having a local man representing each State, in active co-operation with the Federal Bureau, many of the objections now urged against the Washington Department will soon be overcome. It will be far easier to overcome these minor objections in administration than to create a new bureau. Stability of organization, and the following out of a fixed policy through many years until the funds advanced are actually returned with interest is essential to development on a large scale. If the States put up one-half the money for such work, closer scrutiny of all expenditures will be had, and funds will not be invested in particular projects for political reasons only.

It would thus appear from the very close relation between irrigation and water power, from the very wide range over which electric power can be transmitted, and from the innumerable legal and practical complications, that it is extremely unlikely that the United States will ever turn over to the individual States her undeveloped water powers; also, it is believed that such action would lead to far more confusion than the policy of co-operation for the immediate development of both power and irrigation possibilities without regard to State lines.

Conclusion.

In closing this discussion on the question of State and National water laws, the writer wishes to urge on all engineers the importance of studying the question from the standpoint of what the laws ought to be, to stimulate investments in works for the utilization of water. The lawyer is paid to protect private rights, and to ascertain what the law is. His experience and training totally unfit him for the important duty of framing constructive water legislation. Engineers, perhaps, are more responsible for the present chaotic condition of water laws than the members of any other profession.

The present law is very unsatisfactory, from the standpoint of the investor. There must be some Governmental agency to which he can go to ascertain what the law is, how much water in any stream is already vested, the extent of surplus water belonging to the public, and what procedure must be followed in establishing a definite right to such surplus. He must also have assurance that when his money is invested, the State or the Nation will protect him in his right, and not leave him to wage an unequal fight in the Courts, with perhaps a great commonwealth of the Union or with one of the National bureaus interested in the stream. Mr. Lewis.

If it is proper at this time, and it is within the province and scope of the work of the Society, the writer would like to suggest that, as soon as convenient, the President of the American Society of Civil Engineers be authorized and directed to appoint a commission of engineers to draft a law dealing with interstate and National problems relating to water, and to publish such draft as soon as possible for comment and criticism. At the close of such discussion, perhaps the same commission should be called on to reconsider such proposed law in the light of any criticisms offered, and present the same at some appropriate time for adoption by the Society. Such bill could then be forwarded to Congress and might serve as the foundation for needed constructive water legislation.

A programme of this nature would greatly stimulate research and study along this line, promote discussion, and serve to educate the public as to the necessity for reform in both State and National water laws.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ECONOMIC ASPECT OF SEEPAGE AND OTHER LOSSES IN IRRIGATION SYSTEMS.

Discussion.*

By E. G. Hopson, M. Am. Soc. C. E.†

E. G. Hopson, M. Am. Soc. C. E. (by letter).—In closing the discussion on this paper, the writer notes that some who have taken part therein have expressed the opinion that in certain localities, with which they are familiar, conditions would not warrant the cost of protecting ditch systems against seepage losses by lining. In the paper the writer has only attempted to indicate that, under certain conditions, such lining is warranted, and that the conditions requiring lining are tending to increase each year, judging the matter entirely from the economic standpoint. Mr. Hopson.

In making this statement it is well understood that, with low-priced crops, reasonably tight soils and subsoils, and an abundant water supply, the extra expense of placing concrete in the canals is unwarranted. A few years ago such action would have been considered unwarranted in almost every case. The writer has endeavored to illustrate the changes in opinion that are being forced by more straitened conditions in water supply, as irrigation development proceeds and the available lands are being more fully utilized, giving as an example an important project, the feasibility of which has mainly depended on the economic use of the water supply, only possible by the avoidance of waste by seepage in the beds of the canals.

In placing concrete lining in irrigation laterals, attention is called to the necessity of making careful selection of the ingredients of the concrete. The writer has had experience in ditch lining which has not fulfilled the purpose of minimizing seepage, mainly due to the use of

* Continued from January, 1913, *Proceedings*.

† Author's closure

Mr. Hopson. coarse aggregates in the concrete itself. Coarse aggregates give a concrete which, possibly, is entirely satisfactory, from the standpoint of strength and durability, but relatively porous. In selecting the aggregates, it is believed to be very important to obtain well-graded mixtures containing sufficient fine material to furnish a dense water-tight concrete, using the term "water-tight" in the relative sense. A little care in making the selection and grading can generally be expected to give good results.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

SPECIFICATIONS FOR METAL RAILROAD BRIDGES MOVABLE IN A VERTICAL PLANE.

Discussion.*

By Messrs. P. J. Reich and B. R. Leffler.†

P. J. Reich, M. Am. Soc. C. E. (by letter).—Mr. Leffler's "Specifications for Bridges Movable in a Vertical Plane" are quite complete. In the following the writer suggests a few insertions which would eliminate possible misunderstandings; questions the advisability of a number of unusual limitations; and brings up one general question—the greatest power to be assumed as being exerted by the motors, in the calculations of the machinery to hold it, a point on which there seems to be no uniformity. Mr. Reich.

Manner of Bidding.

The classification of the materials in detail for payment is very good, and is to be recommended in preference to the more general classifications that are frequently used. It would be advisable in Paragraph 3 to exclude from the machinery—in addition to the electric motors—all gas and steam engines, together with their equipments. These are usually purchased by the bridge contractor, ready for operation. As the weights are not ordinarily at hand for estimating, it would be preferable and more convenient to place a lump-sum price on each, with its equipment.

Paragraphs 6 and 7.—It would generally be preferable to omit "sockets" from Paragraph 7, and include them in Paragraph 6, with wire ropes, as they are furnished and attached to the rope by the wire rope sub-contractor.

Paragraph 8.—It is probably the intention of Paragraph 8, taken in connection with Paragraph 3, that attached machined parts, such as

* Continued from February, 1913, *Proceedings*.

† Author's closure.

Mr. Reich. cast-steel sleeves, bronze bushings, etc., for movable joints in links and similar members, be weighed separately and classified as machinery. A note added to this paragraph would prevent any misunderstandings.

Similarly, in Paragraph 10, it is advisable to add that "tread-plates" are to be included with the segmental and horizontal girders.

Several minor items not mentioned for payment are: Operator's house with its fixtures, machinery housings, and lumber, each of which should be paid for at separate lump-sum prices. (Structural steel in operator's house, or machinery housings should be classed with structural steel.)

Gas-pipe railings, which are usually expensive, should be paid for at a separate price per pound, the size of railing being specified.

Operating Machinery.

Paragraph 36.—"Trunnions, pins, and shafting more than 3½ in. in diameter shall be of forged structural steel." It is questionable whether enough good comes from this low limitation to warrant the additional expense and delays incident to getting them. The writer has used the rolled rounds up to 5 and 5½ in. with entire satisfaction.

Paragraph 39.—In limiting pinions to forged steel, it would be necessary to make exceptions for those of unusual size, allowing them to be cast. Exceptions should also be made to the cutting; and cast teeth should be required when the pinions mesh with cast teeth, as would be the case where pinions mesh with the racks.

Paragraph 48.—"Nuts shall be secured by split pins put through the bolt." This may be interpreted in two ways: As a general note for all bolts; or only for those where nuts are apt to work off. To make it a general requirement would be an unnecessary expense, and it is suggested that they be used in special cases only where requested by the engineer.

Paragraphs 55 to 59.—Proportions, etc., of Keys.—As many of the bridge companies have standards for keys, to which they are accustomed, and which approximate very closely those outlined, it would be preferable to make the specifications a little more flexible, permitting their use when acceptable to the engineer. This should not be objectionable, as the theoretical requirements are taken care of under Paragraph 56. Gib heads on keys are not always reliable, as they frequently break off. Further, gib-head keys are not as simple to make and fit as those without. Much better results would be obtained by requiring keys to be placed so that they can be backed out, using the gib-head keys only where this cannot be done.

Paragraph 62.—"Keys shall be held in place by set-screws." This seems to be an unnecessary general requirement, in view of the fact that the machinery ought to be in charge of a competent man who

should go over it frequently enough to detect any key which might be working out. It is doubtful if a key that would work loose would be held long by a set-screw. Where keys might readily drop out, should they become loose, as in the case of a vertical shaft, effective means should be used for holding them. Mr. Reich.

Paragraph 68.—This paragraph limits the use of Babbitt metal to shafts 3 in. or less in diameter. When the working pressures do not exceed 500 lb. per sq. in., Babbitted bearings are entirely satisfactory for the larger ones, are easy running, economical, and simple to replace.

Paragraph 77.—"Dust covers shall be provided for principal bearings, in particular for trunnions" (where shown on general plans). It is very desirable, from a contractor's point of view, to have these at least indicated where required, and it is suggested to modify this paragraph by adding the words: "Where shown on general plans." This is also referred to when discussing Paragraph 93a.

Paragraph 86.—Taking care of the effect of key-ways, could also be excepted for overhanging shafts, in which case the maximum moment is at the center of the bearing where no key-ways are cut.

Paragraph 88.—This paragraph limits gear teeth to 20° involute. The differences and relative advantages of the 15 and 20° involute teeth over one another are not sufficient to exclude one or the other. In the past, the 15° involute was the more commonly used, and hence more bridge shops are equipped to cut them. A change means a large expense, which is out of proportion to the advantages gained.

Paragraph 92.—This paragraph limits the angle of thread of worm gearing to 20° as a minimum. As far as wear and efficiency are concerned, an angle of thread of from 13 to 15° gives very satisfactory results, and ought to be permitted. The use to which the worm gearing is put also governs, to some extent. If the span were coasting, a wheel in the main train of gearing would of necessity turn the worm. The writer has no data as to just what this angle would be.

Paragraph 92a.—The writer has frequently used 24-toothed worm wheels, and knows of some 21-toothed wheels in service, working satisfactorily. It would seem that 28 as a minimum might be high.

Paragraph 93a.—The contractor should be asked to furnish only such safety guards as are shown on the general plans. To include all, from a general clause such as this, may mean much or little, depending on the engineer in charge. The addition of these, or changes in their location after the detailed plans are completed, almost invariably costs the contractor more, in his drafting department, than it would to take care of it entirely at the bridge site after the erection of the work. It would be far more satisfactory to the contractor if the safety guards referred to in Paragraph 93a, the dust covers in Paragraph 77, and railings in general which are not shown on the

Mr. Reich. plans, were taken care of entirely by the purchaser. In the end, it would be the more economical to the purchaser, would ordinarily avoid delays, by removing uncertainties, and they would unquestionably be more suitably placed.

Workmanship.

Paragraph 121.—Although it is not definitely permitted, the writer would infer from this paragraph that racks with their pinions are to have cast teeth.

Motors.

Paragraph 156.—"* * * the motor shall be of ample capacity to move or turn the bridge at the required speed. All machinery parts shall be designed with sufficient strength to resist the greatest pressure which can be exerted by the motor." It is very desirable that a method of obtaining the horse-power of the motors from the torque curves of Paragraph 31, be fixed by the specifications; and, further, if any additional allowance of power is to be made for a margin of safety, it should be stated. The fixing of this is essential to another important requirement: The designing of the machinery to enable it to resist the greatest pressure which can be exerted by the motor.

The assumptions to be made for calculations to take care of holding the motor are many, and it would be advisable to have it fixed by the specifications. It is very desirable to have it definitely fixed how the over-loads which the motors must stand should be taken care of in the machinery, giving permissible increases in the unit stresses, etc. These over-loads for which the motors are good are large, as given in Paragraph 173.

The design of the machinery is largely dependent on the assumptions made for these two items, they being somewhat dependent on each other. It is very desirable, therefore, that they be given explicitly in the specifications.

A common method of taking care of the two items is to make the horse-power of the motors sufficiently large, using the maximum torque, as given by Paragraph 31, and the maximum speed as a basis for getting the horse-power. All machinery is proportioned for the normal torque of the motor, and, as low unit stresses are used for the machinery, there is no doubt that the machinery will hold the motor. There may be times, if electrically operated, that the higher starting torque may increase the unit stresses used, but this can be regulated by having the automatic circuit breaker arranged so that the time for opening or closing will approximate the calculated time.

More accurate methods may be devised, depending on the actual torque curves of the motors, but this is objectionable, as the motors are usually the last thing ordered and wanted; and, further, it would delay the progress of the design too much to get these at the start.

Paragraphs 166 to 168.—The last sentence of each of these paragraphs requires the contractor to furnish a work-bench with a full set of machinist's tools, etc. It would be preferable to have these furnished by the purchaser, as the exact needs could then be taken care of, this being dependent, to some extent, on the operator and the proximity of a good machine shop. It is objectionable to the contractor to furnish these, as the final cost is an uncertainty, because of uncertain requirements; further, when they are furnished with the bridge, they are usually carried away by some one before bridge is taken over. Mr. Reich.

Paragraphs 197 and 216.—Any special requirements by city codes for the electric installation, if the bridge is subject to city authority, or any Government or other harbor special requirement for channel lights and signals, should preferably be furnished by the purchaser at the time of asking bids. The purchaser is usually in a better position to get this information, and it means a saving of considerable trouble to each of the bidders.

Paragraphs 160 to 220.—It would be very desirable, from a contractor's point of view, if Paragraphs 160 to 220, covering power and operator's house equipment, were divided into three distinct and separate clauses: One for steam-engine equipment, one for internal-combustion motor equipment, and one for electrical equipment. The specifications are nearly in this order at present. The separation is desired for simplicity and to make it unnecessary to pick out the parts that might be interpreted as being required.

These specifications cover a first-class bridge which would be used on a main line of a railroad over a stream where navigation requires frequent opening. A supplement might profitably be added, waiving certain restrictions in the design and workmanship of the machinery and equipment, for economical purposes, for bridges not on a main line, or for those which are not opened frequently.

In conclusion, the writer would add that one essential feature to the success of movable bridges, to which not enough attention is ordinarily given, is the erection. Many of the troubles ordinarily credited to other sources would be eliminated entirely if a good competent machinist, familiar with this class of work, were added to the regular forces, to superintend the installation of the machinery.

B. R. Leffler, M. Am. Soc. C. E. (by letter).—The writer desires to express his appreciation of the discussion of his paper, and offers the following notes on some of the points brought out. Mr. Leffler.

As a result of the discussion and the writer's experiences since the paper was written, the specifications have been revised and are presented in separate form.

Taking up Mr. Smith's discussion:

Paragraph 55.—The writer sees no objection to other forms of keys, provided they are of the right proportions. However, a tapered key

Mr. Leffler. with a moderate slope, and held by a set-screw through the hub, is quite adequate. The specifications have been modified to allow the use of other keys.

Paragraph 92.—The writer believes that the only justification for using a worm gear, in the bridges covered by the specifications, is for the purpose of getting a large reduction in a limited space where a train of gears could not be used. For this case, efficiency is of more importance than the holding power of the worm. In swing bridges the worm has been used to raise the ends of the bridge; here the holding power is of much importance.

Paragraph 136.—The writer does not believe it advisable to look to the handbooks of wire rope makers for the proper diameter of sheaves for counterweight cables. It seems that a rational theory which gives safe design is preferable to tabulated empirical data.

The writer's knowledge concerning wire rope is not extensive enough to say just where a rope will break. Apparently, it should break where it starts to bend over the sheave. Over the sheave, the frictional resistances of the rope on the sheave groove probably strengthens the rope. Over the sheave, the rope is subject to radial forces which modify the conditions.

Paragraph 140.—The character of the stresses in gear teeth at their contact is unknown. Line contact with sliding is found; the intensity of the stresses between any two bodies pressed together on a line is not known. This is the problem of designing a roller for heavy loads. The best pair of meshing gears, according to the writer's experience, is that of a wooden-toothed bull-wheel and cast-iron pinion.

Teeth designed by the Lewis formula seem to be satisfactory. In the writer's experience, teeth seem to break instead of wearing out. Recently, in the horizontal rack of a rolling lift bridge under the writer's care, several teeth were broken on account of the sudden application of the brakes; the material was cast steel.

For other points by Mr. Smith, the specifications have been modified.

Referring to Mr. Mercer's discussion, the following views are presented:

Paragraph 21.—In rolling bascule bridges, pure rolling is not found. For the full roller, a flange is used; for the segmental roller, teeth in the track. Hence, there is some sliding friction. Moreover, there is considerable deformation, which becomes partly permanent. In one case under the observation of the writer, the rolling structure refuses to close without the application of extra torque; this is due to permanent deformation at the end of the track. Some rolling structures are not precise in their movements; the free end is forced sidewise into its closed position by guides. In lieu of more exact knowledge, the co-

efficient of $\frac{1}{12}$ is a workable one. A coefficient for pure rolling is added. Mr. Leffler.

The coefficient for stiffness of wire rope is not well known. The efficiency of a wire rope belt over two pulleys may be equal to 96%, including journal friction. *Eytelwein's formula for stiffness is $R = \frac{Kd^2}{D} P$, in which K is a constant; d is the diameter of the rope, and D is that of the sheaves, both in inches. P is the weight lifted. The pull, which lifts the weight, must equal $R + P$. K can be taken at $\frac{1}{3}$. The specifications are revised accordingly.

The ratio of the diameter of the sheave to that of the rope in the Halsted Street lift at Chicago is 90. After 18 years of service, the ropes were still in use. A high ratio should be used to limit the friction, and consequent wear, between the wires. A paragraph has been added to the specifications to cover this. According to Rankine, for ratios above 140, the effect of rigidity is negligible.

Paragraph 22.—Journals are not subject to the high pressure found in trunnions; the coefficient at starting is not as great. Again, there is some play in a train of gears, and the motor overcomes the initial journal friction before the initial trunnion friction comes into play.

Paragraph 23.—It is desirable to define clearly the line between the designer and the fabricator. The writer accepts a revision. However, the writer has found it necessary to make specifications for the designers.

Paragraph 36.—The writer recommends the use of a softer phosphor-bronze instead of a harder steel.

Paragraph 168.—The writer has one swing bridge operated by a gasoline engine. The rail-locks are of the sliding-sleeve type, and are actuated by the engine. The gasoline engine is not a suitable type of prime mover for a bascule bridge; it does very well for a lift, if placed on the moving span, and in this position it can be made to operate end-locks for rails and ends of span.

For bridges of the type herein discussed, sliding rail-locks may be dispensed with. The writer has devised a lock which holds securely the ends of the rails, and bridges the gap between the shore rail and the bridge rail, and no machinery is necessary for its operation.

Paragraph 192.—The writer is not prepared to give a specification for submarine cables. It is the practice of the railroad company with which he is connected to furnish the cable, because such cable contains the wires for signal, telegraph, light, and telephone services, as well as power wires for the bridge.

Paragraph 197.—The writer believes that the National Electric Code represents good practice. Every bridge has an operator's house, or

* References: Church's "Mechanics of Engineering," 1890, p. 193.
Kennedy's "Mechanics of Machinery," 4th Ed., Art. 80.

Mr. Lefller. power station, in which part of the electric equipment is installed. Obviously, the installation in the house should be first class, from a fire-protection standpoint.

There are many excellent provisions in the Code which should be used in any first-class electric installation. The character and installation of circuit breakers, fuses, switches, motors, etc., are important. To cover all these points in a specification would be a useless repetition of what is already recognized as first-class practice. Hence, a mere reference to the Code should be sufficient.

Paragraph 266.—The writer believes that the suggestions for phosphor-bronze are excellent. The physical qualities of phosphor-bronze suitable for heavy bearings seem to be somewhat unknown.

The writer has under observation a trunnion which became abraded. The phosphor-bronze showed a permanent set of 0.06 in. on a 1-in. cube under a 100 000-lb. load. Another, and similar, case has occurred. Hence, it seems that a somewhat softer phosphor-bronze is more suitable. The specification suggested is adopted.

The writer has not commented on all the points raised by Mr. Mercer. Nearly all those omitted have been adopted in the revised specification.

Referring to Mr. Reich's discussion, the suggestions are excellent, and the writer has adopted most of them in the revision.

Paragraph 68.—Babbitt metal gives good results if properly poured, but where accuracy of alignment and correct relative position of the machinery parts are to be maintained, bronze and brass linings are better.

Paragraph 88.—The proper angle of obliquity in involute teeth is still unsettled, although the matter has been before the American Society of Mechanical Engineers for several years. For an angle less than 20°, it is necessary to modify the outline, to avoid interference. Even for 20°, a slight change must be made for a pinion of 17 teeth or less meshing with a rack.

The writer finds that the special gear-cutting concerns readily undertake to cut gears of any obliquity without much extra expense. *The present trend is toward an angle of 20° as a standard.

Paragraph 92.—For worm gears, used in bridges of the character specified, efficiency is of more importance than holding power. An angle of thread of 20° is about the lowest limit for a loss of 30 per cent.

Paragraph 156.—The writer has attempted in the revision to clear up the question of designing for excess torques.

The use of the term "Horse-power" is somewhat misleading. It is better to specify the capacity of a motor by naming the vari-

* In the "American Machinist Gear Book," pp. 20 to 26, the subject is fully discussed.

Mr. Leffler.

able torque it should exert for a given time, say twice the normal variable torque shown on the torque curves for a cycle of operation.

The writer believes that the torque curves should be worked up as quickly as possible. This is necessary for the correct determination of the gear ratio, and the capacity of the storage battery, if this is intended as the source of power.

Regarding a specification for bridges of minor importance; it is the writer's experience that such bridges give the most trouble in maintenance. Any power-operated bridge should be opened at least once a week, in order to keep the parts in smooth running condition. A movable bridge, to fulfill its purpose, must be in first-class condition continually, even if seldom moved. After a number of years of experience in maintaining movable bridges, it is the writer's opinion that it is impossible to keep a power-operated bridge too efficient.

The writer begs to submit a demonstration of the formula for the bending of wires, given in Paragraph 38. This is largely of academic interest. The Trenton Iron Company, in its "Wire Rope Book," gives a formula which is intended to serve the same purpose.

In attempting to reduce the Trenton formula, it was found to be erroneous. *Further investigation led to the derivation of the formula given in Paragraph 38. In the deduction a slightly different limiting value of the angle is adopted. This gives a closer approximation.

The Deduction of the Value of R in Paragraph 138.—

Proposition.—A wire is bent over a sheave through a small angle. Determine the least radius of curvature, R, of the bend.

Let Fig. 4 represent the conditions, considerably distorted for purposes of reasoning.

For practical purposes, θ is the angle between the straight parts of the wire.

Let I = Moment of inertia of the wire.
M = Bending moment in the wire at any point.
E = Modulus of elasticity.
W = Pull on wire.
S = Length of curve, measured from the y axis.
$RM = EI$.

Now, by calculus,

$$R = \left(\frac{ds}{dx}\right)^3 \cdot \frac{dx^2}{d^2y} \quad \text{.......................(1)}$$

$$M = W\,(y - x \cot.\ \alpha) \sin.\ \alpha \quad \text{..................(2)}$$

Now, as α is to be a large angle, it will be assumed to be not less than 65 degrees. In Equation (1) $\dfrac{ds}{dx}$ can never exceed cosec. α in value, or be less than 1: its cube will be practically a constant, say b.

* Private correspondence, dated May 21st, 1912, with Mr. William Hewitt.

Mr. Leffler. Then

$$R = b\,\frac{dx^2}{d^2\,y} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

hence,

$$b\,\frac{dx^2}{d^2\,y}\;W\,(y \sin.\,\alpha - x \cos.\,\alpha) = EI.$$

$$\frac{d^2\,y}{dx^2} = \frac{b\,W\,(y \sin.\,\alpha - x \cos.\,\alpha)}{EI}$$

Put $\dfrac{b\,W \sin.\,\alpha}{EI} = a$, and $\dfrac{b\,W \cos.\,\alpha}{EI} = h$, then

$$\frac{d^2\,y}{dx^2} = ay - hx \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

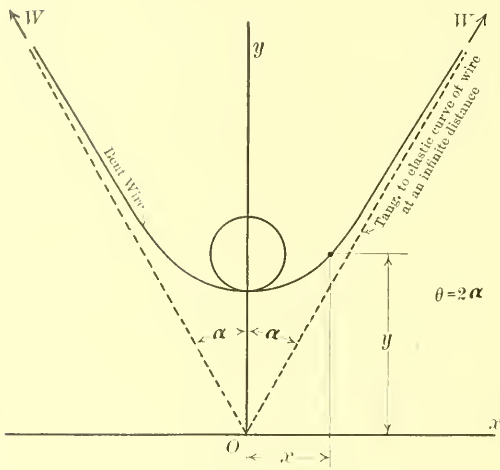


FIG. 4.

Integrating Equation (5),

$$y = c_1\,e^{\,x\sqrt{a}} + c_2\,e^{\,-x\sqrt{a}} + \frac{hx}{a} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$e = 2.718 +$$

Now,

$$\frac{hx}{a} = x \cot.\,\alpha,$$

hence

$$y = x \cot.\,\alpha + c_1 e^{\,x\sqrt{a}} + c_2 e^{\,-x\sqrt{a}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Equation (7) is the equation of the elastic curve.

$$\frac{dy}{dx} = \cot.\,\alpha + c_1\,\sqrt{a}\;e^{x\sqrt{a}} - c_2\,\sqrt{a}\;e^{-x\sqrt{a}} \qquad \ldots\ldots\ldots (8)$$

For $x = 0$, $\dfrac{dy}{dx} = 0$. Equation (8) becomes $0 = \cot.\,\alpha + c_1\,\sqrt{a} - c_2\,\sqrt{a}$

For $x = \infty$, $\dfrac{dy}{dx} = \cot.\,\alpha$. Equation (8) becomes $\cot.\,\alpha = \cot.\,\alpha + c_1\,\infty$.

Hence, $c_1 = 0$, and $c_2 = \dfrac{\cot. \alpha}{\sqrt{a}}$. Mr. Lefller.

Equation (7) now becomes

$$y = x \cot. \alpha + \frac{\cot. \alpha \, e^{-x \sqrt{a}}}{\sqrt{a}}$$

For $x = 0$, $y = \dfrac{\cot. \alpha}{\sqrt{a}}$, this is the value of y where the curve crosses the y axis.

Equation ($1a$) now becomes, for R at the y axis, by putting $x = 0$, and $y = \dfrac{\cot. \alpha}{\sqrt{a}}$ in Equation (3),

$$R = \frac{b}{\cot. \alpha \sqrt{a}} = \frac{\sqrt{b}}{\cos. \alpha} \sqrt{\frac{EI \sin. \alpha}{W}}.$$

This is the least value of the radius of curvature.

As the limiting values of $\dfrac{ds}{dx}$ are 1 and cosec. $\alpha = 1.10$, we will assume as an average value for $\dfrac{ds}{dx}$, 1.05, hence, as $\left(\dfrac{ds}{dx}\right)^3$ was called b, $\sqrt{b} = 1.08$.

Again, as α is never less than 65°, the average value of its sine can be taken at 0.953.

$$\text{Then } R = \frac{1.08 \times 0.976}{\cos. \alpha} \sqrt{\frac{EI}{W}} = \frac{1.054}{\cos. \alpha} \sqrt{\frac{EI}{W}}.$$

Again,
$$I = \frac{\pi d^4}{64}.$$

hence
$$R = \frac{1.054 \sqrt{\pi} d^2}{8 \cos. \alpha} \sqrt{\frac{E}{W}} = \frac{d^2}{4.27 \cos. \alpha} \sqrt{\frac{E}{W}}.$$

It is close enough to take

$$R = \frac{4 d^2}{17 \cos. \frac{\alpha}{2}} \sqrt{\frac{E}{W}}.$$

REVISED SPECIFICATIONS FOR RAILROAD BRIDGES MOVABLE IN A VERTICAL PLANE.

Scope. 1.—These specifications are intended to cover bascule bridges, which are such as rotate about a horizontal axis; and vertical lifts, which are those in which successive positions are parallel.

Deck. 2.—The Contractor shall place and fasten permanently all ties, rails, guards, and other deck material. Usually, the Railroad Company will furnish all the deck material except rail-locks, rails which must be fabricated to fit locks, and special devices to hold the deck in place.

Responsibility. 3.—If complete general proposal plans are furnished to the Contractor, he shall be responsible for only the material and character of the workmanship and installation. However, for any parts of the design not covered by the proposal plan, and for which the Contractor is to make a design, he shall be wholly responsible.

Electrical Equipment. 4.—Unless otherwise specified, the electrical equipment shall include all electrical parts up to, and including, the switch-board. Electrical equipment carrying the current to the switch-board from the source of power will be covered by separate contract.

5.—The specifications of the New York Central Lines for Steel Railroad Bridges, for 1910, shall apply to movable bridges, except as noted herein.

Manner of Bidding.

Parts Classified as Machinery. 6.—Drums, cylinders, eccentrics, trunnions, and their cast supports, shafting, pistons, gear wheels, racks, boxings, bearings, couplings, disks, cast sheaves and wheels, worm gearing, valves, pins about the axis of which the connecting members rotate, whistles, ram screws, end bridge-locks, rail-locks, indicators, cranks, axles, hooks, wrenches, and similar parts of machinery which require machine-shop work, shall be classified as machinery and be paid for at a common price per pound. Electric motors, or other prime movers, are not classified as machinery. (11).

Sheaves. 7.—The large sheaves of vertical lift bridges the webs and diaphragms of which are built up with plates, angles, and rivets, shall be paid for at a separate price per pound of finished weight, including casings and fastenings to trunnions.

Air Compressor Boilers. 8.—Air compressor tanks and steam boilers shall be paid for at a separate price.

Wire Ropes and Cables. 9.—Wire ropes and cables and their sockets shall be paid for at a separate price per pound.

Pins, Levers, etc. 10.—The pins, equalizing levers, and cable attachments to the trusses and counterweights shall be paid for at a separate price per pound.

Structural Steel Parts. 11.—Structural steel supporting the machinery proper, counterweight frames, counterweight trusses, steel in operator's house, towers,

and links, shall be classified as structural steel and be paid for at the same price per pound as for the span itself. Attached machine parts, such as sleeves, bushings, etc., shall be classified as machinery. (6).

12.—The operator's house, except for the structural steel therein, shall be paid for on a lump-sum basis. Operator's House.

13.—Structural steel which can be fabricated by the common shop methods, as punching, reaming, drilling, shearing, planing, etc., as is usually done for stationary structures, shall be classified as structural steel and be paid for at the same price per pound as for the span itself.

14.—Segmental girders in rolling bascule bridges and the horizontal girders on which they roll shall be paid for at a separate price per pound. This does not include any bracing, floor system, or other structural members which may be attached. Track plates shall be included. Segmental Girders.

15.—Hand-rail shall be paid for at a separate price per pound. Hand-Rail.

16.—Electrical equipment, such as wiring, switch-boards, controllers, lights, blow-outs, cut-offs, solenoids, switches, motors, etc., shall be paid for on a lump-sum basis. Electrical Equipment.

17.—Internal-combustion engines and steam engines shall be paid for on a lump-sum basis. Engines.

18.—Cast-iron parts used in counterweights shall be paid for at a separate price per pound.

19.—Concrete in counterweights shall be paid for at a price per cubic yard in place.

20.—It is to be understood that if any extra parts are needed, or any question arises, all difficulties shall be settled on the pound price basis as quoted and accepted for the parts in question. Extra Parts, etc.

General Details in Designing.

21—Self-centering and seating devices shall be used on the free ends of the moving span. Holding and forcing-down devices shall be used for the free ends of each truss. Self-Centering Devices.

22.—Designs for bridging the gap between the shore rails and moving rails shall be furnished by the Railroad Company. Loose rails will not be allowed. Rail-Locks.

23.—Air buffers shall be furnished at the free ends of the moving span. Air Buffers.

24.—The counterweights shall be easily adjustable. Usually, this shall be done by adding or taking away cast-iron parts, or small concrete blocks. Counterweights.

25.—Metal stairways, with 1½-in. hand-rail, shall be provided, for access to the machinery, trunnions, and counterweights. The pipe shall weigh 2.68 lb. per ft. Stairways.

Girders in Rolling Bridges.

26.—The reinforcements of webs in the segmental girders and track girders of rolling bridges shall be symmetrical about the center planes of the webs. The center planes of the segmental webs shall coincide with the corresponding center planes of the webs of the track girders. That part of an outstanding leg of an angle, which is beyond the outside face of the upstanding leg, shall not be considered as reinforcement. The width of contact between the segmental webs (including the reinforcements) and the back of the tread plates, shall be equal to the width of the corresponding contact in the track girder.

Coefficients of Friction for Moving Span and Attached Parts.

27.—In calculating the resistances to be overcome by the machinery, the resisting forces shall be reduced to a single force acting between the pinion and operating rack, or in the operating cable. In determining this force, the following coefficients shall be used in starting the span, and, except for the stiffness in cables, shall be reduced one-half after motion is begun. (44):

For trunnion friction	$\frac{1}{8}$
For rolling friction of bridges having rollers with flanges, or built-up segmental girders	$\frac{1}{12}$
For stiffness of wire rope per 180° of bending: d = diameter of rope, in inches; D = diameter of sheaves, in inches	$\frac{d^2}{3D}$

Coefficients of Friction.

28.—For a solid cast roller, without flanges, in contact with one surface only, the coefficient of rolling friction shall be taken equal to $\frac{15}{1\,000\,r}$, in which r is the radius of the roller, in inches.

29.—In figuring the machinery losses between the operating rack or operating cable and the motor, the following coefficients shall be used:

For the efficiency of any pair of gears, journal friction not included	0.99
For journal friction	0.06
Losses in a worm gear for an angle of thread of 20° or more	0.30

30.—For sliding friction between plane surfaces intermittently lubricated, such as guides on tower posts, the coefficient of friction shall be taken equal to 0.08.

Time to Open.

31.—The time to open the bridge after the ends are released shall be approximately as specified on the proposal drawing.

Inertia.

32.—The force necessary to overcome the inertia and produce acceleration and retardation for the time of opening shall be considered. The machinery shall be capable of stopping the bridge in 6 sec.; for this purpose, the coefficient of friction in the friction brake shall be taken at not less than 25 per cent.

33.—In calculating the dead-load stresses in the moving structural parts, for the various positions of the open bridge, such stresses shall be increased 25% as allowance for impact. For stationary structural parts (as towers, and supporting girders in rolling bridges), to which moving structural parts are attached, or on which such parts roll, 15% of the static stress shall be added as impact. Impact in Structural Parts

34.—In structural steel parts, where a percentage of the dead load or static stress is added for impact, the unit stresses for stationary structures shall be used; the impact percentages are an allowance similar to that provided by an impact formula for stationary railroad bridges.

35.—In structural members subject to reversal on account of the motion of the span, the effect of reversal shall be neglected. The member shall be designed for the stress giving the larger section. For riveted connections, the number of rivets shall be increased 25% over that required for the static stress plus impact stress. Reversal of Stress.

36.—The allowance for impact in trunnions, cables, cable attachments, machinery parts, and structural parts supporting the machinery, is taken care of by lowered unit stresses. Impact for Machinery Parts, etc.

37.—The wind pressure to be assumed in proportioning the machinery or moving parts shall be 15 lb. per sq. ft. on the exposed surfaces of the moving parts as projected on any vertical plane. The machinery shall be strong enough to hold the moving parts in any position for this pressure, and be capable of opening the bridge in the specified time at 10 lb. per sq. ft. wind pressure. Wind Pressure.

38.—The structure shall be strong enough to resist a wind pressure of 15 lb. per sq. ft. on any surface as projected on any vertical plane for any open position of the moving span. The structure shall be strong enough to resist a wind pressure of 25 lb. per sq. ft. when the moving span is in the closed position.

39.—The least wind pressure to be assumed on the floor of the moving span shall be $2\frac{1}{2}$ lb. per sq. ft. for any position of the span. For the vertical lift, this shall be taken as acting throughout the movement. Least Wind Pressure.

40.—On the ordinary open-floor bridge with ties, the exposed surface to wind shall be taken equal to 80% of a full quadrilateral having a width equal to the distance from center to center of trusses and a length equal to that of the moving span.

41.—The Contractor shall make complete detailed drawings of the machinery, so that any other shop can take them and duplicate the machinery. No reference to patterns or individual shop practices will be considered in lieu of the complete drawings. These drawings shall show a general outline of the assembled machinery. The drawings shall be made on tracing cloth, each sheet 24 by 36 in. in outside dimensions. These drawings shall become the property of the Railroad Company on the completion of the job. Detailed Drawings.

Outline Drawing of Machinery. 42.—The Contractor shall furnish an outline drawing of the machinery, on which are shown the forces acting on the gear teeth, the twisting moment and bending moment on shafts, and other necessary information for checking the strength of the machine parts. A tabulation of the formulas and methods of calculation shall be shown complete enough to allow them to be checked.

Torque Curves. 43.—The Contractor shall show by a drawing of curves the torque to be exerted by the motor or prime mover, as follows:

1. A torque curve for acceleration and retardation;
2. A torque curve for the frictional resistance;
3. A torque curve for any unbalanced condition of the structure;
4. A torque curve for the wind load;
5. A torque curve showing the greatest combination of resistances acting at any one time.

44.—In figuring the friction at starting (this being twice the running friction), no acceleration of the moving mass shall be considered. This friction shall be considered as reduced to the running friction in the first second after the power is applied. (27).

Capacity of Wires, etc. 45.—If the Contractor is to furnish the design for the electrical equipment, such as wiring, switches, etc., he shall show by a curve the current required by the motor to overcome the various resistances. This is for the purpose of checking up the carrying capacity of wires and other parts, and determining the storage battery capacity.

Center of Gravity. 46.—The Contractor shall check the location of the center of gravity of the moving span, including all parts attached thereto, and also the location of the center of gravity of the counterweights, including counterweight girders and trusses, by computations based on accurate weights calculated from shop plans. He shall submit duplicate sketches and copies of these computations accompanied by weight bills to the Railroad Company for approval.

Hand Operation. 47.—All bridges shall be equipped with hand-operating mechanism. The number of men and the time required to operate shall be estimated on the assumption that the force one man can exert on a lever is 40 lb. with a speed of 160 ft. per min. developing about $\frac{1}{5}$ h.p. For calculating the strength of the machinery, the power of one man shall be assumed as 125 lb.

Operating Machinery.

48.—The parts shall be simple in design, and easily erected, inspected, adjusted, and taken apart. The fastenings shall hold the parts in place securely after they have been set.

Kind of Material. 49.—Rolled or forged steel shall be used for bolts, nuts, keys, cotters, pins, axles, screws, worms, piston rods, trunnions, and crane hooks, if any.

50.—Trunnions, pins, and shafting more than 4½ in. in diameter, shall be of forged structural steel. Shafting 4½ in. or less in diameter may be of cold-rolled steel.

51.—Forged or cast steel shall be used for levers, cranks, and connecting rods.

52.—Cast steel, or forged steel, shall be used for couplings, end shoes, racks, toothed wheels, brake wheels, drums, sheaves, and hangers where the supported weight will cause tensile stresses. Large sheaves may be built of structural steel.

53.—Pinions shall be made of forged steel and cut from the solid metal, unless such pinions are too large for forgings.

54.—Sockets used for holding the ends of wire ropes shall be forged without welds from the solid steel. The equalizing levers, connecting the ropes to the counterweights, or moving span, shall be of forged steel.

55.—Cast iron may be used in boxes for shafts 2 in. or less in diameter, and which obviously carry light loads. Other boxes shall be of cast steel. Cast Iron.

56.—Cast iron may be used in eccentrics, cylinders, pistons, fly-wheels, and parts of motors which are usually made of cast iron. Cast iron shall not be used for any trunnion or axle supports.

57.—Phosphor-bronze, brass, and Babbitt metal shall be used for the bushing or lining of journal bearings and other rotating or sliding surfaces, to prevent seizing. Metal for Bushings.

58.—Phosphor-bronze, only, shall be used for bushing for the trunnions of bascule and lift bridges, or in any large bearing carrying heavy loads.

59.—The bushings for large bearings, such as for trunnions and similar parts, shall be held from rotating in their casings. The force tending to cause rotation shall be taken as one-eighth of the load on the trunnion or bearing, and as acting at a tangent to the surface between the back of the bushing and casing; this force shall not be considered as counteracted by any frictional resistances between bushing and casing. It shall be practicable to take out the bushing when the trunnion is slightly lifted.

60.—Castings which are to be attached to rough unfinished surfaces shall be provided with chipping strips. The outer unfinished edges of ribs, bases, etc., shall be rounded off, and inside corners filleted. Castings.

61.—Bolts and nuts, up to 1½ in. in diameter, shall have U. S. Standard V-threads. Nuts and exposed bolt heads shall be hexagonal in shape, and each nut shall be provided with a washer. If the nut will come on an inclined surface, a special seat, the top surface of which is at right angles to the bolt, shall be cast or built up to receive the nut. Bolt heads which are countersunk in castings shall be square. Bolts and Nuts.

62.—Nuts which are subject to vibration and frequent changes of load shall have locking arrangements to prevent the gradual unscrewing of the same. If double nuts are used for that purpose, each nut shall be of the standard thickness. Nuts subject to vibration shall be further secured by split pins through the bolt.

Screws. 63.—Screws which transmit motion shall have square threads.

Tap-Bolts, Set-Screws, etc. 64.—Tap-bolts and stud-bolts shall not be used, except by special permission.

65.—Set-screws shall not be used for transmitting torsion to shafts or axles.

Collars. 66.—Collars shall be used wherever necessary to hold the shaft from moving horizontally. Each collar shall have at least two set-screws at an angle of 120 degrees.

Shaft Couplings. 67.—Shaft couplings, unless of the flexible kind, shall be of the flange type, or split muff with bolt heads and nuts countersunk.

68.—For large shafts, couplings such as are used for rolling mill shafting may be used.

69.—Couplings shall be keyed to shaftings.

Keys. 70.—If practicable, hooked and tapered keys shall be used. The taper shall be $\frac{1}{8}$ in. per ft. The width of the key shall be one-fourth of the diameter of the shaft; the height, at mid-section of the tapered length, shall be three-fourths of the width. The length of the hook, measured parallel to the shaft, shall be equal to the width of the key.

71.—If tapered keys are not practicable, parallel-faced keys of about the above proportions shall be used.

72.—Tapered keys shall bear on top, bottom, and sides; parallel-faced keys shall bear on sides only.

73.—The length of any key shall be not less than that of the hub.

74.—The foregoing dimensions are approximate. The shape of the key shall be such as to have unit stresses in shear and bearing not exceeding those allowable in the table.

75.—If practicable, the keys and grooves shall be made so that the keys may be backed out.

76.—Keys shall be sunk in grooves in both hub and shaft. The depth of a groove shall be such that the bearing will not exceed the allowable unit stress.

Set-Screws, etc. 77.—Keys shall be held by set-screws or equivalent means. In vertical shafts, bands clamped about the shaft, or other devices, shall be placed below the key.

Hub. 78.—If practicable, the length of the hub shall be not less than two diameters of the shaft, and its thickness not less than one-third of the diameter of the shaft. The hub shall have a light driving fit on the shaft.

79.—The groove in the hub shall be made on the center line of an arm.

80.—Hubs shall be bored truly at the center of the wheel.

81.—For trunnions and similar parts, which are designed chiefly for bending and bearing, the keys, key-ways, and bolts shall be designed to hold the trunnions from rotating. The force tending to cause rotation shall be taken at one-fifth of the load on the trunnion, and shall be taken as acting at the circumference of the trunnion. Keys in Trunnions.

82.—Journals shall be proportioned to resist, not only the various stresses to which they are subjected, without exceeding the permissible fiber and bending stresses, but also to prevent a tendency to heat and seize. Journals.

83.—Divided journal and trunnion bearings shall be used, and the cap shall be fastened to the base with turned bolts recessed into the base. The nuts and heads shall bear on finished bosses cast on the bearing. There shall be $\frac{1}{8}$ in. clearance between the lining of the base and the cap or its lining, to allow for expansion.

84.—Steel bearings carrying steel shafts or journals shall be lined with bronze or brass. If shafts are 3 in. or less in diameter and of a slow motion, Babbitt metal may be used. Bearings of steel on steel for moving surfaces will not be allowed. Bushings.

85.—In cast-iron boxes carrying light shafts, no lining is needed. Boxes.

86.—The bearings of shafts shall be placed as near to the points of loading as possible.

87.—The foot-steps of vertical shafts shall be of axle or tool steel, and shall run on bronze disks.

88.—Provision shall be made for the effective lubrication of journals, or any other sliding surfaces. Closed oil or compression grease cups shall be used. Grooves shall be cut in the surface of the trunnion to provide for the proper distribution of grease or oil. Grease and oil cups must hold the lubricant for any position of the moving parts. Lubrication.

89.—The grooves in large trunnions shall approximate to a **U** shape; the size shall be such that a wire $\frac{5}{16}$ in. in diameter may lie wholly within the groove. The edge of the **U** shall be rounded to a radius of $\frac{1}{4}$ in. Grease Grooves.

90.—The grooves shall be straight, running parallel to the axis of the trunnion. They shall be not less than three in number, and located so that all parts of the bearing surface of the bushing will be swept by the contained lubricant, in an opening, and in a closing of the bridge. The grooves must allow of being cleaned with a wire.

91.—In any trunnion bearing, or similar heavy bearings, strong compression grease cups shall be used for the grooves. Grease Cups

92.—Oil and grease ducts shall be located so that the lubricant will flow by gravity toward the bearing surface.

93.—Dust covers shall be provided for the principal bearings, in particular for trunnions where shown on plans. Dust Covers.

Shaft Supports and Couplings.

94.—Line shafts, extending from the center of the bridge to the end, shall not be continuous, but shall be connected with claw couplings. Each length of shafting shall rest in not more than two bearings, with the couplings close to the bearings.

95.—If shaft supports are connected to the floor-beam, in bridges having long panels, intermediate supports shall be used; these shall be adjustable, and are intended merely to prevent the shaft from sagging.

Equalizing Gears.

96.—Equalizing gears or devices shall be used to insure equal action at the pinions and operating racks.

Unsupported Length of Shafts.

97.—The unsupported length of shafts shall not exceed $L = 80\sqrt[3]{d^2}$ for shafts supporting their own weight only; $L = 50\sqrt[3]{d^2}$ for shafts carrying pulleys, gearing, etc., where L = length of shaft between center of bearings, in inches; and d = diameter of shaft, in inches.

98.—Line shafts connecting machinery at the center to that at the ends shall run at fairly high speed. The speed reduction shall be made in the machinery near the end.

99.—In designing circular shafting, trunnions, and axles, the greatest unit fiber stress in tension or compression due to bending shall be calculated by the following formula:

$$f = \frac{32}{\pi d^3}\left(\frac{3}{8} M + \frac{5}{8}\sqrt{M^2 + T^2}\right)$$

Formulas for Shafts.

100.—The maximum unit shear shall be calculated by the following formula:

$$S = \frac{16}{\pi d^3}\sqrt{M^2 + T^2}$$

101.—In these formulas, f = unit fiber stress in tension or compression; S = unit shear; d = diameter of shaft; M = the simple bending moment; and T = the simple twisting moment.

102.—If a shaft, trunnion, or axle has one key-way cut, at the section where the maximum stresses occur, f and S shall be increased by one-sixth; if two key-ways are cut, increase by one-fourth. If the shaft is enlarged through the hub, this does not apply.

Distance between Shaft Support.

103.—In calculating the bending moment on shafts, trunnions, and journals, the distance from center to center of bearings shall be taken.

Style of Gear Teeth.

104.—Gear teeth shall be of the involute type, with an angle of obliquity of 20 degrees. The roots below the clearance line shall be filleted.

105.—The width of the teeth may be as great as four times the pitch, but not more, except for wheels running at a very high velocity, as in motors where abrasion is to be considered.

Strength of Beveled Gear Teeth.

106.—In estimating the strength of teeth in bevel wheels, the pitch at the middle section shall be taken.

107.—For the purpose of setting gear teeth accurately in the field erection, the pitch circle shall be scribed on the ends of the teeth. Pitch Circle.

108.—Worm gearing, for transmitting power, shall have an angle of thread not less than 20 degrees. The worm shall run in oil. A bronze or brass collar shall be used at the end of the worm and at the end of the wheel axle, to take care of the end thrust. The wheel shall be of bronze. If a nut engages the worm, the nut shall be of bronze. Worm Gearing.

109.—Worms which are to be used for actuating signals, indicators, or other minor parts, may have an angle of thread less than 20 degrees.

110.—Worm wheels shall have not less than twenty-six teeth. Number of Teeth in Worm Gears.

111.—Pinions shall have not less than fifteen teeth. Number of Pinion Teeth.

112.—For the purpose of keeping down the wear between the individual wires in counterweight rope, the diameter of the sheave shall be at least ninety times that of the rope. Diameter of Sheaves.

113.—Machinery parts near which workmen may be while the parts are in motion, shall be designed so that safety guards may be added. These guards will be furnished and installed by the Railroad Company. Safety Guards.

Counterbalancing, Operating Ropes, and Attachments.

114.—Wire rope shall be made by a manufacturer approved by the Engineer. Wire Ropes and Cables.

115.—The counterbalance ropes shall be of plow-steel wire, and shall consist of six strands, of nineteen wires each, laid around a hemp center.

116.—Ropes shall be laid up in the best manner, and shall be thoroughly soaked in an approved lubricant during the process of manufacture.

117.—The counterbalance ropes shall be made from wire which has been tested in the presence of an inspector designated by the Engineer, and which, for sizes from 0.076 to 0.150 in. in diameter (the limiting values used in counterbalance ropes), exhibits the following physical properties:

a.—The tensile strength shall be not less than 225 000 lb. per sq. in. for wire from 0.150 to 0.126 in. in diameter, nor less than 230 000 lb. for wire from 0.125 to 0.101 in. in diameter, nor less than 235 000 lb. for wire from 0.100 to 0.076 in. in diameter.

b.—The total ultimate elongation, measured on a piece 12 in. long, shall be not less than 2.4 per cent.

c.—The number of times a piece 6 in. long can be twisted around its longitudinal axis without rupture shall be not less than 1.4 divided by the diameter, in inches.

d.—The number of times the wire can be bent 90°, alternately to the right and to the left, over a radius equal to twice its diameter, without fracture, shall be not less than six. This test shall be made

in a mechanical bender constructed so that the wire actually conforms to the radius of the jaws and is subjected to as little tensile stress as possible.

Ultimate Strength of Cables.

118.—The rope shall be made in one piece, if possible, Its breaking strength, as determined by test described in Paragraph 121, shall be not less than

4 900	lb.	if	$\frac{1}{4}$	in.	in	diameter.
11 800	"	"	$\frac{3}{8}$	"	"	"
20 600	"	"	$\frac{1}{2}$	"	"	"
32 400	"	"	$\frac{5}{8}$	"	"	"
45 000	"	"	$\frac{3}{4}$	"	"	"
70 000	"	"	$\frac{7}{8}$	"	"	"
79 200	"	"	1	"	"	"
100 800	"	"	$1\frac{1}{8}$	"	"	"
120 600	"	"	$1\frac{1}{4}$	"	"	"
148 000	"	"	$1\frac{3}{4}$	"	"	"
173 000	"	"	$1\frac{1}{2}$	"	"	"
200 000	"	"	$1\frac{5}{8}$	"	"	"
230 000	"	"	$1\frac{3}{4}$	"	"	"
264 000	"	"	$1\frac{7}{8}$	"	"	"
297 000	"	"	2	"	"	"
325 000	"	"	$2\frac{1}{8}$	"	"	"
374 000	"	"	$2\frac{1}{4}$	"	"	"
465 000	"	"	$2\frac{1}{2}$	"	"	"

119.—In case the physical qualities of the rope, or its individual wires, fall below the values cited above, the entire length from which the test pieces were taken shall be replaced by the manufacturer with a new length, the physical qualities of which come up to the specifications.

120.—The dimensions of the sockets shall be such that no part under tension shall be loaded higher than 65 000 lb. per sq. in. when the rope is stressed to its ultimate strength, as named above. The sockets must be attached to the rope by a method which is reliable and will not permit the rope to slip in its attachment to the socket.

121.—In order to show the strengh of the rope and fastenings, a number of test pieces, not more than 10% of the total number of finished lengths which will ultimately be made, nor less than two from each original long length, and not more than 12 ft. long, shall be cut, and shall have sockets, selected at random from those which are to be used in filling the order, attached to each end. These test pieces are to be stressed to destruction in a suitable testing machine. Under this stress the rope must develop the ultimate strength given in Paragraph 118.

122.—If, in testing, slipping in the sockets should occur, then the method must be changed until slipping is avoided. The sockets themselves shall be stronger than the rope with which they are used; if one should break during the test, then two others shall be selected and attached to another piece of rope and the test repeated; and this process shall be continued until the inspector is satisfied of their reliability, in which case the lot shall be accepted. If, however, 10% or more of all the sockets tested break at a load less than the minimum ultimate strength of the rope, as given in Paragraph 118, then the entire lot shall be rejected.

123.—The length of each rope, from inside of bearing to inside of bearing of socket, shall be determined, and a metal tag having the said length stamped thereon shall be attached securely to the rope. Length of Rope.

124.—One-third of the wire rope connections, selected at random, shall be tested (after attachment to the ropes are made) up to 0.4 of the ultimate strength of the rope. If any connection is weak, the remainder of the connections shall be tested. The weak connections shall be rejected and replaced. Not less than four connections shall be tested. Testing Rope Connections.

125.—The manufacturer shall provide proper facilities for making the tests, and shall make at his own expense all the tests required. Tests shall be made in the presence of an inspector who represents the Engineer. Facilities for Testing Rope.

126.—Ropes shall be shipped in coils the minimum diameter of which is at least thirty times that of the ropes, and they shall be uncoiled for use by revolving the coil, not by pulling the rope away from the stationary coil. Shipment of Rope in Coils.

Workmanship.

127.—For the parts of the operating machinery of movable bridges which are usually exposed to the weather, the finish shall be confined to the bearing, rotating, and sliding surfaces, and wherever it is required to produce accurate fits and precise dimensions.

128.—Equalizing levers in rope connections shall be neatly finished, and shall conform to the dimensions shown on the drawings.

129.—Castings shall be cleaned, and seams and other blemishes removed.

130.—Drainage holes, not less than $\frac{3}{4}$ in. in diameter, shall be drilled in places where water is likely to collect.

131.—Unfinished bolts may have a play of $\frac{1}{16}$ in. in the bolt holes. Turned bolts must have the diameter of the shank at least $\frac{1}{16}$ in. larger than the diameter of the threaded portion, and must have a driving fit in the bolt hole. Play in Unfinished Bolts.

132.—The backs of racks and surfaces in contact shall be planed. Racks and Contact Surfaces.

Sheaves. 133.—The bearing surfaces, at the junction of the cast rim and structural parts in built-up sheaves, shall be planed to give full contact throughout the circumference of the sheave.

Grooves in Sheaves. 134.—The grooves in the circumference of sheaves carrying wire ropes shall be turned to a radius that will fit the rope. This is to be done after the sheave is completely assembled and permanently riveted up.

Tread Plates. 135.—The top and bottom of the tread plates and surfaces in contact in rolling bridges shall be planed to fit. A full bearing must be made.

136.—The periphery and the ends of teeth which mesh with shrouded teeth shall be planed, and the pitch line shall be scribed thereon.

Finishing of Trunnions, etc. 137.—Journals and trunnions shall be turned with a fillet where the section changes. Journals shall have a collar at each end. Trunnions and journals 8 in. in diameter and greater shall have a hole, $1\frac{1}{2}$ in. in diameter, bored through on the longitudinal axis. Journals, trunnions, and bushings must be polished after being turned. The use of a cutter which trembles or chatters will not be allowed.

138.—The joints between the caps and bases of journal and trunnion bearings shall be planed. The ends of the bases and surfaces in contact with the supports shall be planed. Bolt holes for holding the cap to the base and for holding the base to its support shall be drilled.

Grooves. 139.—The grooves in the surfaces of trunnions or similar large bearings shall be machine cut. Chipping and filing will be allowed only for removing small inequalities. The grooves shall be smooth, especially the rounded corners.

Hubs. 140.—Hubs of wheels, pulleys, couplings, etc., shall be bored to fit close on the shaft axle. If the hub performs the function of a collar, the end next to the bearing shall be faced. Holes in hubs of toothed gear wheels shall be concentric with the pitch circle.

Cut Gears, etc. 141.—The periphery of gear wheels shall be turned. Gear wheels and racks which are part of the train which actuates the moving span shall be cut. Other gears, except beveled gears, shall be machine moulded.

142.—If any moulded gears are shrouded, chipping or other means shall be used at the junction of the shrouding and teeth to insure proper meshing.

Beveled Gears. 143.—Beveled gears shall be cut. The cutting shall be done by a planer having a rectilinear motion to and from the apex of the cone. Rotating milling cutters shall not be used.

144.—Threads on worms, and the teeth of worm wheels shall be cut, and shall fit accurately. Point contact shall be avoided.

145.—Any two surfaces which slide, or roll, or bear on each other, shall be planed or turned to fit.

Assembling of Machinery.

146.—Machinery parts shall be assembled on the supporting members in the shop, and shall be aligned and fitted, and holes in the supports drilled, and with the members in correct relative position. The members shall be match-marked, both to the supports and to each other, and re-erected in the same relative position.

Holes for Sheaves for Vertical Lift Bridges.

147.—The holes in the girders and columns for the bolts connecting the main sheave bearings to their supporting girders shall be drilled from the solid through cast-iron or steel templates on which the bearings were set and accurately lined when the holes in the bearing were bored. The bolt holes and the bolts shall be turned to the same diameter, and the bolts shall be driven to place without injury to them, the bearings, the girders, or the columns.

Testing of Trunnion Bearings.

148.—Trunnions shall be turned to a diameter of $\frac{1}{50}$ in. less than that of the bushing. Before shipping, the trunnions shall be placed in their bearings and given two full rotations. If any grinding or hard turning is found, it must be remedied. The tests shall be made in the presence of the Railroad Company's inspector.

Facing of Couplings.

149.—Faces of flange and split muff couplings shall be planed to fit. The couplings shall be keyed to the shaft.

150.—A special effort to secure good workmanship on keys and keyways shall be made.

Coating of Surfaces.

151.—Machined surfaces shall have a coating of white lead applied to them.

152.—Machinery which is of the regular standard manufactured type, such as steam, gasoline, electric motors, pumps, air compressors, etc., shall be guaranteed by the manufacturer as to efficiency, and shall be subject to the approval of the Engineer. Motors shall be tested to prove that they fulfill the specified requirements and develop the desired speed, power, and torque.

153.—The rating of a prime mover shall be the horse-power determined by the brake test.

Unit Stresses.

Normal and Excess Loads.

154.—Machinery parts shall be designed for the normal loads used in determining the torque curves, using the unit stresses herein specified. For the excess torque specified for the prime mover, twice the normal unit stresses will be allowed. (178) (205) (208).

Braking.

155.—If brakes act through the machinery, the unit stresses produced by braking shall not exceed by more than 50% those caused by the normal torque of the prime mover.

156.—The following unit stresses, in pounds per square inch, shall be used for parts in which main stresses are not increased by impact:

Stresses in One Direction.

Material.	Tension.	Compression.	Fixed Bearing.	Shear.
Machinery steel...	9 400	$9\,400 - 40\dfrac{l}{r}$	11 000	6 200
Structural steel....	8 500	$8\,500 - 36\dfrac{l}{r}$	......	5 600
Steel castings.....	7 000	$8\,000 - 35\dfrac{l}{r}$	......	5 000
Phosphor-bronze..	6 600	.....	......	4 600
Cast iron.........	3 000	8 000	......	3 000
	Shear on keys....	4 900		
	Bearing on keys..	8 800		

157.—The maximum unit tension in plow-steel cables shall be one-sixth of ultimate. The maximum unit tension is equal to the direct unit stress plus the extreme fiber unit stress in the individual wire due to bending over the sheave.

Reversal of Stresses.

158.—For stresses which are reversed at the rate of ten or more times per minute, use one-half of the above unit stresses.

159.—If wire rope is bent over a sheave, the bending stress and permissible load on the rope shall be calculated as follows:

Let P = the total pull or permissible load on the rope, in pounds;
K = extreme unit fiber stress in greatest individual wire;
E = modulus of elasticity = 28 500 000;
a = cross-sectional area of rope, in square inches;
d = diameter of thickest wire, in inches;
D = diameter of sheave to center of rope, in inches;
S = greatest unit tension allowable;
α = angle of helical wire with axis of strand;
β = angle of helical strand with axis of rope;
c = diameter of rope.

$$\text{Then } K = \frac{Ed \cos^2 \alpha \cos.^2 \beta}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$P = a\left(S - \frac{Ed \cos.^2 \alpha \cos.^2 \beta}{D}\right) \quad \ldots\ldots\ldots\ldots (2)$$

For rope having six strands of nineteen equal wires each,

$$P = a\left(S - \frac{1\,800\,000\,c}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

because $\cos.^2 \alpha \cos.^2 \beta = 0.95$, $d = \dfrac{c}{15}$.

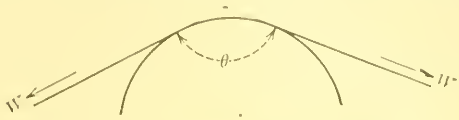
160.—For haulage rope, six strands of seven wires each, take $d = \dfrac{c}{9}$.

161.—If a rope is in contact with a sheave over a small arc, the actual radius of curvature may be greater than that of the sheave. (See figure.)

Let R = The actual radius of curvature;

θ = the angle between the directions of the rope;

W = pull on individual wire, equal to P divided by the number of wires if all wires are of equal diameter.



$$\text{Then } R = \frac{4\,d^2}{17 \cos. \frac{\theta}{2}} \sqrt{\frac{E}{W}}$$

162.—If R is greater than the radius of the sheave, $2R$ should be used in place of D in Formulas (1), (2), and (3). The formula is only valid for values of θ between 130 and 180 degrees.

163.—The strength of cut gear teeth shall conform to the following formula, one tooth only taking pressure: Strength of Gear Teeth.

$$P = fpb \left(0.154 - \frac{0.912}{n}\right) \frac{600}{600 + v}, \text{ in which}$$

P = pressure on tooth, in pounds;

f = permissible unit stress = 17 000 lb.;

p = pitch, in inches;

b = face or breadth of tooth, in inches;

n = number of teeth in gear;

v = velocity on pitch circle, in feet per minute.

164.—The strength of machine-moulded teeth shall be calculated by the foregoing formula, taking f = 15 000 lb.

165.—The foregoing formula is for involute teeth having an angle of obliquity equal to 20 degrees.

166.—The pressure, in pounds per linear inch, on rollers at rest shall be, for rolled and cast steel, 600 d, where d equals the diameter of the roller, in inches. Pressure on Rollers.

Unit Stresses for Bearing on Rotating and Sliding Surfaces.

167.—Maximum bearing values for rotating and sliding surfaces, in pounds per square inch:

Bearings on which the speed is slow and intermittent:

	Pounds per square inch.
Trunnion bearings on bascule bridges: machinery steel on phosphor-bronze	1 500
Wedges: Cast steel on cast steel or structural steel	500
Screws which transmit motion on projected area of thread	200

	Pounds per square inch.
For ordinary cases, parts moving at moderate speeds: Hardened steel on hardened steel........	2 000
Hardened steel on bronze........................	1 500
Tool steel (not hardened) on bronze.............	900
Structural steel on bronze......................	600
Cast iron on structural steel...................	400
On cross-head slides, speed not exceeding 600 ft. per min....................................	50

168.—In order to prevent heating and seizing at higher speeds, the pressure on pivots or foot-step bearings for vertical shafts and journals shall not exceed:

On pivots.................. $p = \frac{40\ 000}{n\,d}$ per square inch.

On journals.............. $p = \frac{300\ 000}{n\,d}$ per square inch.

Where n = number of revolutions per minute;
d = diameter of journal or pivot, in inches.

169.—For crank pins and similar joints with alternating motion, the limiting bearing values given in the above formula may be doubled.

Stresses on Rollers.

170.—Permissible pressure, in pounds per linear inch of roller in motion:

For cast iron .... $p = 200d$
For steel castings .... $p = 400d$
For machinery steel .... $p = 500d$
For tool steel .... $p = 800d$
For hardened tool steel.................. $p = 1\,000d$

Where p = pressure per linear inch of roller,
and d = diameter of roller, in inches.

171.—The foregoing values are for rollers and bearing surfaces of the same material; if rollers and bearing surfaces are of different materials, the lower value shall be used.

Power Equipment.

General Requirements.

172.—The kind of motor best adapted to any particular case depends on local conditions, and should be left to the judgment of the Engineer.

Mechanical Power.

173.—If the bridge is operated by mechanical power, the motor shall be of ample capacity to move or turn the bridge at the required speed. No matter what mechanical power is used, all bridges shall also be provided with hand-power operating machinery.

174.—Friction brakes, to be operated by hand or foot, shall be provided where the motor is located in the operator's house. They shall have sufficient capacity to stop or hold the moving span in any position, under all conditions. Friction Brakes.

175.—If mechanical power of any kind is to be used for operating a movable bridge, a suitable house shall be provided for the operator. The house shall be of such dimensions as required for the purpose for which it is to be used. It shall be placed in a position where the operator can observe the signals and see the approaching vessels and trains, and with enough windows of sufficient size, so that his view will not be obstructed. If the operator's house is above or below the floor of the bridge, suitable steel or iron stairs with railings shall be provided to lead from the floor of the bridge to the floor of the operating house. The house shall be of fire-proof construction, consisting of a steel frame, steel floor joists and a fire-proof floor. If the house contains motors and machinery, the floor shall preferably consist of steel plates, but, if the motors are located elsewhere, the floor between the joists may be of concrete construction. The sides and roof shall be of metal, concrete, or any other non-combustible material. The hand-rail for stairways and other places shall be made of 1½-in. gas pipe. Operator's House.

176.—Whenever climatic conditions require it, provision shall be made for heating the operator's house. If steam power is used, the house shall be heated by a steam coil or radiator fed from the boiler. If electric power is used, the heat may be supplied by electricity. If gasoline is used, or any other power which cannot be utilized for heating, a coal, wood, petroleum, or gas stove, as directed by the Engineer, shall be provided. Heating of Operator's House.

177.—A whistle, having a bell 5 in. in diameter and 12 in. long, shall be installed complete. If operated by air, the compressor and air tank shall conform to the following specifications: The compressor shall be motor driven, the motor and compressor being on one frame, and geared. All working parts shall be completely enclosed, and self-lubricating. The compressor shall have a piston displacement of from 25 to 30 cu. ft. per min. when working against a tank pressure of 90 lb. per sq. in. The compressor shall be provided with strainer, and automatic governor and switch, in order that the compressor may start and stop automatically at any predetermined tank pressure. The air receiving tank shall be 36 in. in diameter and 8 ft. long, or of equal capacity. The tank shall be galvanized, and good for a working pressure of 100 lb. per sq. in. It shall be provided with pressure gauge and pigtail, pop-valve and drain cock, and have standard flanges bushed for 1½-in. pipe. The Contractor shall furnish all pipe, pipe fittings, and valves, and all shall withstand a working pressure of 100 lb. per sq. in. Whistle.

178.—A prime mover shall be capable of exerting twice the greatest torque shown on the torque curves for the normal loads. (154). Greatest Torque.

Steam Power.

Steam Engine. 179.—If a steam engine is used, it shall consist of a double-cylinder, reversing engine, the piston speed of which shall not exceed 200 ft. per min.; it shall develop the desired power and speed with a steam pressure of 50 lb. per sq. in. The engine shall be connected to the operating machinery by an approved friction clutch, arranged so that the moving and locking machinery can be operated alternately or stopped without stopping the engine.

Steam Separator. 180.—In the steam supply pipe, and close to the steam chest, shall be placed a steam separator. This separator, under test with quality of steam as low as 66%, shall show an average efficiency of 85% in five tests.

Boilers. 181.—The steam shall be generated by one or two upright, tubular boilers, each of which shall have twice the capacity of the engine. The boilers shall be designed for a steam pressure of 150 lb. per sq. in., and adapted to the kind of fuel specified by the Engineer; they shall be of open-hearth steel, in accordance with the specifications for boiler plates appended hereto. They shall be encased in asbestos and covered with Russia iron.

182.—The boilers shall also be in accordance with the specifications of the Mechanical Department of the Railroad Company, and shall conform to the civil laws.

Flues of Boilers. 183.—Vertical boilers shall have submerged flues at the top.

Horse-power of Boilers. 184.—The total horse-power of the boilers shall be twice that of the engine, and shall be computed by the following rule: Calculate the inside area of the tubes, the area of the tube sheet next to the fire, and the sides of the fire-box where this is in contact with the fire. Take the sum of these areas in square feet and divide by fifteen. The intention is to allow 15 sq. ft. of heating surface per horse-power. At least $\frac{1}{3}$ sq. ft. of grate surface shall be provided per horse-power.

Equipment of Engine-Room. 185.—The engine-room shall be provided with a steel water tank of sufficient capacity; a duplex, steam feed-pump; and an injector for each boiler, with necessary pipes and connections for feeding boilers separately or together; steam water-lifters with necessary strainers, flexible hose, and piping to lift the water from the river into the tank; a coal hoist and a steel coal-bin of sufficient capacity. The engine-room shall be provided with a suitable indicator for recording the positions of the moving span in turning and locking.

Internal-Combustion Engines.

Gasoline Motor, etc. 186.—If an internal-combustion engine is used, one of the most substantial kind shall be selected, the maximum piston speed of which shall not exceed 500 ft. per min. The engine shall have a reversing gear provided with approved friction clutches, to be operated by a hand-wheel. The countershaft connecting the engine with the operat-

ing machinery shall be provided with disengaging couplings, arranged so that the moving and locking machinery can be operated alternately and in either direction without stopping the engine. Motors of 10 h.p. or more shall be started by compressed air. The fuel tank shall be located outside of the engine-house. The engine-room shall be provided with indicators for recording the positions of the moving span, and lifting and locking apparatus.

187.—For bridges which are to be opened at intervals of 10 min. or less, and about six times per hour, the engine shall be water-cooled. For longer intervals, the engine shall be air-cooled. For this purpose, the outside cylinder shall have deep flanges about which a forced circulation of air is maintained by a fan. Engine Cooling.

188.—The ignition shall be of the jump-spark kind in which the secondary coil is made up on each spark plug as part of it, so that a low voltage current of not more than 10 volts will be sufficient. Ignition.

189.—Two extra igniters and two extra crank-pin brasses shall be furnished. Extra Parts.

Electric Equipment.

190.—The electric equipment shall conform to the Standardization rules of the American Institute of Electrical Engineers, as adopted June 21st, 1907, or subsequent revisions. A. I. E. E. Rules.

191.—The National Electric Code shall apply to the electric construction and installation, except as may be noted hereinafter. N. E. C.

192.—The quality of the wires and insulation shall conform to the specifications of the Railway Signal Association, as revised and adopted October, 1911. (See Volume 8 of the *Proceedings* of that Association.) Wires and Insulation.

193.—Any motor under test shall develop the required horse-power and torque at the armature shaft. Characteristic curves showing the results of the test shall be furnished by the manufacturer. Tests.

194.—Motors shall be tested for one-half their normal voltage between the brushes. An efficiency curve shall be shown.

195.—Motors, generators, automatic circuit breakers, solenoids, brakes, and other electric mechanism shall be tested at the factory by the manufacturer in the presence of the Railroad Company's inspector.

196.—If the motor is enclosed in a case, as mill motors are, small openings of sufficient size shall be provided in the case for the inspection, removal, and replacing of brushes.

197.—One cast-steel cut gear, bored and key-seated for attachment to the countershaft, shall be furnished with the motor. The gear and pinion shall be covered by a sheet-steel or malleable-iron split gear case, supported by the motor frame and completely covering the gear and pinion. An opening, with a hinged cover, shall be provided in

the gear case for inspection and oiling. The gear ratio shall be such that the full speed of the countershaft will not be more than 125 rev. per min.

Motor Pinions. 198.—Motors shall have a forged-steel cut pinion, out of one piece, keyed to the end of the armature shaft and secured by a lock-nut.

Spare Motor Parts. 199.—For each size of motor furnished, the Contractor shall supply the following spare parts: One armature, one field coil, one pinion, one gear, and one set of brushes. These parts shall be finished and fitted in such a manner as to admit of being installed in their respective places without further fitting or adjustment.

Mounting Motors. 200.—The motors shall be mounted in such a manner as to admit of easy access for inspection and repairs; they shall be supported securely by brackets or suitable foundations.

201.—If the machinery and motors are on the moving span, they shall be capable of being operated satisfactorily in any position of the span.

Housing of Motors. 202.—Motors must be housed in weather-proof metal housing. This housing must be large enough to allow the inspection and oiling of the motor. It must be readily removable so that access to the motor may be obtained. No metal in this housing shall be less than No. 16, U. S. Standard, gauge; it shall be galvanized.

D. C. Motor. 203.—Direct-current motors and generators shall be of the railway series, or mill, interpole type, water-proof, with slotted-drum armature and form-wound armature coils. They shall be of a standard commercial type in common use.

Testing of Motors. 204.—The rating of a direct-current motor is the horse-power output at the armature shaft which gives a rise of temperature above the surrounding air (referred to a room temperature of 25° cent.) not exceeding 90° cent. at the commutator and 75° cent. at any other part after a continuous run of 1 hour at its rated voltage, on a stand with the motor covers removed and with natural ventilation. The rise in temperature is to be determined by thermometer, but the resistance of no electric circuit in the motor shall increase more than 40% during the test.

Excess Motor Loads. 205.—Direct-current motors shall be capable of exerting continuously for two cycles twice the normal torque shown on the torque curves for the moving span and machinery. One cycle is an opening and closing of the bridge in a specified time. (154) (224) (226).

A. C. Motors. 206.—Alternating-current motors shall be of the three-phase, induction type, with slip-rings, rotor-wound, 25 or 60 cycles, and 220 or 440 voltage, unless otherwise specified. The resistance for varying the speed shall be in series with the rotor circuit, and be such as to affect evenly all three phases. Motors of 5 h.p. or less may be of the squirrel-cage type.

207.—Alternating-current motors shall show, in a run for heat test, the following maximum temperature rises above 25° cent. for

the surrounding room: for continuous run under a nominal load, 40° cent.; for 2 hours' run under 25% overload and a 1 min. run under 50% overload 55° cent.

208.—Alternating-current motors shall be of rugged construction. The sum of the starting torques of the motors shall be at least equal to twice the greatest torque shown by the torque curves for the bridge operating machinery. The pull-out torque shall be at least equal to 1¼ times the starting torque. (154) (224) (226).

209.—The controllers for motors shall be located in the operating-house. The controllers shall be of the reversing drum type, or flat type, with magnetic blow-out, and shall be capable of varying and maintaining the speed of the motors throughout the entire range desired, without injurious sparking, and without shock due to sudden variation in speed. The controllers shall be capable of doing their work for the usual loads, and excess loads, that may come upon the motors, with a temperature rise not exceeding that specified for the motors. Controllers.

210.—The controller shall have a sufficient number of notches or steps, such that the minimum or maximum motor torque will not differ by more than 10% from the average torque required for uniform acceleration. Controller Steps.

211.—One controller shall be furnished for the operation of the main motors, one for rail-lock motors, and one for bridge-lock motor. These controllers shall be designed so that the operation of any motor can be cut out by pulling a switch on the switch-board, without affecting the operation of any of the other motors.

212.—The controllers for the two main motors, if for direct current, shall be of the series-parallel type; or of the type in which the field is varied, as may be done for the interpole type of motor.

213.—The control of motors shall be electrically interlocked with each other and with the signal system, and the bridge shall be controlled in such a way that the end-locks cannot be released until the signals have gone to danger position and the derails are set; and the bridge motor cannot be started until the end-locks have actually been released. In closing the bridge, the control shall be such as to make it impossible for the operator to move the end-locks until the bridge has been completely closed, or to set the signals at safety until the bridge has been closed and the end-locks are in place. Control of Motors.

214.—For currents too large for the usual type of controller, the motor circuits shall be made by contactors mounted on panels or frames. These contactors shall be operated by solenoids which are controlled by a master controller. Master Controller.

215.—For large structures, automatic control may be used, but this is too complicated to be covered by a specification. This should be taken up for special consideration with the Engineer. Automatic Control.

Resistance. 216.—Resistances shall be of the cast-grid type, and of such capacity that the motor can be operated continuously at any point of the controller when developing full-load torque, or for 10 min. when developing 50% over-load torque, without sufficient rise in temperature of the resistance to cause deterioration of any part. The resistance shall be mounted so as to admit of free ventilation and be without injurious vibration.

Electric Brakes. 217.—The main operating motors, rail-lock motors and bridge-lock motors shall be provided with approved post brakes which are held in set position by a spring with such force as to overcome not less than 50% of the maximum torque required. The friction surfaces are to be of materials not affected by moisture. The brakes are to be released by solenoids of ample power and heating capacity whenever the motors are taking current, and are to be set automatically whenever the current fails or is cut off from the motors. Water-proof motors shall be provided with water-proof solenoids. Brakes shall be provided with a foot-switch release for coasting purposes. Means shall be provided for mechanically releasing the brakes when the bridge is to be operated by hand or other equipment.

Emergency Brakes. 218.—An additional emergency brake shall be provided and applied to the main operating machinery. This shall be released by solenoids or motors which shall hold the brake in release as long as the current is applied to the brake motor. Cutting off the current from the solenoids or motors, or any failure of current, will result in the instantaneous application of the brake. This brake will be normally set, but will be released by the operator before starting the bridge, and be held in release during the entire operation unless an emergency condition arises requiring brake power in excess of that offered by the motor brakes, in which case it may be instantly applied by the operator. After the bridge has been closed and traffic has been resumed, this brake will again be applied. This portion of the equipment shall be designed so that it will not be injured if left in release indefinitely. Proper means shall be provided for releasing the brake mechanically when the bridge is to be operated by hand or emergency-power equipment.

219.—The emergency brake circuit shall be independent of the general interlocking system, and there shall be a mechanical interlocking device which will prevent the main leaf motors and the emergency brake from being used one against the other.

220.—The emergency brake switch shall be attached to the controller stand within easy reach of the operator, and proper labels shall be placed back of the switch handle to indicate "Set" and "Released" positions of the brake.

Automatic Cut-offs. 221.—An automatic cut-off or short-circuiting device shall be provided which will throw out the circuit breakers, cut off the current

from the operating motors, and set their brakes when the bridge is 5° from its open position, and its closed position. Spring switches shall be provided which, if closed and held closed, will put the cut-offs out of commission and thus enable the bridge tender fully to close or open the bridge.

222.—The bridge-lock motors, and rail-lock motors, shall be stopped and the brakes set automatically at each end of the travel.

223.—Switches shall be designed to carry not more than 900 amperes per sq. in. of cross-sectional capacity. Size of Switches

224.—Electrical parts, such as wires, switches, etc., shall be designed for the currents which are needed for the excess torques in motors. (205) (208). Excess Current.

225.—Ground connections of ample area shall be provided.

226.—Circuit breakers and fuses shall be designed to act when the current exceeds 2½ times the current required in the motors to exert their greatest normal torque and horse-power for the specified time. (205) (208).

227.—Enclosed fuses shall be used. Fuses.

228.—No wire smaller than No. 10, B. & S. gauge, stranded wire, shall be used. Minimum Stranded Wire.

229.—Wires when installed shall be permanently tagged and numbered so that any wire can be traced from the switch-board to the motors, and to the source of power. Wires to be Tagged.

230.—The feeders shall be protected by a pole-switch fuse and lightning arrester mounted on a non-combustible and non-absorbent insulating base. Lightning Arrester.

231.—A switch, of the quick-break type, shall be provided for each supply wire. Each motor circuit and each light, signal, indicator, or other circuit, shall be provided with switches which are approved by the Railroad Company's Engineer. The switches shall be mounted on an enameled slate panel switch-board (not less than 1¼ in. thick, and free from metallic veins, or flaws) in the operator's house. The switch-board shall be large enough to carry the meters, switches, cut-outs, fuses, etc. Switches, cut-outs, buttons, etc., shall be provided with plates designating their use. Quick-Break Switch and Switch-board.

232.—An automatic circuit breaker shall be placed on the switch-board in the operating motor circuit of the bridge. Each line to the motor, each line to the electric brakes, and each lighting, signal, indicator, or other circuit, shall be protected by enclosed fuses. Automatic Circuit Breaker.

233.—Automatic circuit breakers shall be placed near the top of the switch-board, with the other instruments below.

234.—Any circuit whatsoever shall be protected by fuses, circuit breakers, or equivalent devices, which will insure the excessive current being cut off before any parts are damaged.

Lightning Arresters. 235.—Lightning arresters shall be placed as near as practicable to the parts to be protected, and away from combustible material. A No. 4, B. & S. gauge, wire should be used for the connections; this wire should run in a straight line to a ground plate, and not be connected to any structural parts. To avoid inductive resistances, the wire should not run through a conduit. If a choke-coil is used, it should be thoroughly insulated from the ground and other conductors.

Short Circuiting. 236.—The connections of parts in contact with track shall be such as to allow no short circuiting of track signals.

Protection of Electric Contacts. 237.—Electric contacts shall be protected from the weather or accumulations of dirt.

Coils. 238.—Coils shall be impregnated.

Solenoids, etc. 239.—Solenoids and electrically-operated brakes shall be housed.

Indicators. 240.—The Contractor shall provide and install electric light indicators for the purpose of showing the operator the various positions of the bridge, especially the fully open, entirely closed, nearly open, and nearly closed positions of the bridge, and fully closed and fully open positions of the rail-lock and bridge-locks.

Volt Meter, etc. 241.—A volt meter, ammeter, and watt meter shall be provided on the switch-board.

Ground Detector. 242.—The switch-board shall be furnished with one 2-c.p. lamp for detecting ground, and a 2-c.p. lamp for illuminating the ammeter and volt meter scales.

Lamps for Lighting. 243.—In the operator's house shall be placed ten 16-c.p. lights, and additional lights about the machinery and such other lights as the Engineer may direct. For all lights in the house above ten in number, the Railroad Company will pay the regular market price or furnish them to the Contractor.

244.—Lights of 16-c.p. shall be placed outside at the head and foot of stairways or similar paths.

245.—All lights in the house shall have tungsten filaments. Outside lights shall have weather-proof sockets.

Channel Lights. 246.—The Contractor shall furnish warning and channel lights and signals, in accordance with the U. S. Government requirements, or other harbor requirements. The Railroad Company will furnish a copy of the U. S. Government regulations.

Railroad Signal System. 247.—The Company will furnish and install the railroad signal system, also the master lever and all necessary devices controlling the interlock between this signal system and the bridge as a whole. The Contractor shall furnish and install the necessary devices for interlocking the various parts of the bridge with each other and for connections to the Company's master lever.

248.—Emergency switches shall be provided which will free the various motors from the interlocking, in emergencies. These switches shall be mounted on the switch-board, but covered by a sealed glass case.

249.—Unless the current supply is taken from more than one source, it shall be conducted to the switch-board in two independent conductors, one for the supply, and one for the return current. Current Supply.

250.—Submarine cables, if needed, will be furnished and laid by the Railroad Company. Submarine Cables.

251.—If wires are to be placed in conduits, the conduits shall be of ample size, sherardized, or loricated on the inside. No wire less than No. 12, B. & S. gauge, shall be used. Conduits and Minimum Size of Wire.

252.—Conduits shall be of sufficient size to allow the wires to be easily drawn in. No joints are to be made inside of a conduit. Condulets and factory ells shall be used. Condulets, ells, and conduits shall be sherardized, or loricated inside. Condulets and Factory Ells.

Specifications for Special Metals Used for Machinery Parts.

Steel Castings.

253.—Steel for castings may be made by the open-hearth or crucible process. Qualities of Steel Castings.

254.—All castings shall be annealed, unless otherwise specified.

Phosphorus	0.05% maximum.
Sulphur	0.05% maximum.

255.—Minimum physical qualities, as determined on a standard test specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch	70 000
Elongation: percentage in 2 in.	18
Contraction of area: percentage	25

256.—A test to destruction may be substituted for the tensile test, in the case of small or unimportant castings, by selecting three castings from a lot. This test shall show the material to be ductile, free from injurious defects, and suitable for the purpose intended. A lot shall consist of all castings from the same melt or blow, annealed in the same furnace charge.

257.—Castings shall be true to pattern, and free from blemishes, flaws, or shrinkage cracks. When the bearing surface of any steel casting is finished, there shall be no visible blow-holes exceeding 1 in. in any direction, nor exceeding ½ sq. in. in area. The length of blow-holes cut by any straight line laid in any direction shall never exceed 1 in. in any 1 ft. Flaws in Castings.

258.—No blow-hole exceeding one-half the above dimensions and area will be allowed in any gear tooth, or in the rim at the root of the teeth. Blow-Holes in Gear Wheels.

259.—The correction of defects in castings, by welding electrically by thermit or by similar processes, will not be allowed. Electric Welding.

260.—Large castings shall be suspended and hammered all over. No cracks, flaws, defects, or weakness shall appear after such treatment. Testing of Large Castings.

261.—A specimen (1 in. by ½ in.) shall bend, cold, around a diameter of 1 in., through an angle of 90°, without fracture on the outside of the bent portion.

262.—The number of standard test specimens shall depend on the character and importance of the casting. A test piece shall be cut, cold, from a coupon to be moulded and cast on some portion of one or more castings from each melt or blow, or from the sink-heads (in case heads of sufficient size are used). The coupon or sink-head must receive the same treatment as the casting or castings, before the specimen is cut out, and before the coupon or sink-head is removed from the casting.

263.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether or not the steel is within the limits in phosphorus and sulphur specified in Paragraph 254 concerning chemical properties.

Steel Forgings.

Qualities of Steel Forgings.

264.—Steel forgings may be made by the open-hearth or crucible process:

Phosphorus .... 0.04% maximum.
Sulphur .... 0.05% maximum.

265.—Minimum physical properties as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length.

266.—Tensile strength, in pounds per square inch, 85 000 to 65 000.
Elongation: percentage in 2 in......................... 28

A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of ½ in., without fracture on the outside of the bent portion. The bending may be effected by pressure or by blows.

267.—The number and location of the test specimens to be taken from a melt, blow, or forging shall depend on their character and importance, and, therefore, must be regulated by individual cases. The test specimens shall be cut, cold, from the forging, or full-sized prolongation of the same, parallel to the axis of the forging and half way between the center and the outside; the specimens shall be longitudinal, *i. e.,* the length of the specimen shall correspond with the direction in which the metal is most drawn out or worked. When forgings have large ends or collars, the test specimens shall be taken from a prolongation of the same diameter or section as that of the forging back of the large end or collar. In the case of hollow shafting, either forged or bored, the specimen shall be taken within the finished section prolonged, half way between the inner and outer surfaces of the wall of the forging.

268.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred

by the inspector, shall be used to determine whether or not the steel is within the limit in chemical composition.

269.—Forgings shall be free from cracks, flaws, seams, or other injurious imperfections, and shall conform to the dimensions shown on the drawings furnished by the purchaser, and shall be made and finished in a workmanlike manner.

270.—All forgings shall be annealed.

Machinery Steel.

271.—Machinery steel shall be made by the open-hearth or crucible process. Qualities of Machinery Steel.

Phosphorus................0.05% maximum.
Sulphur...................0.05% maximum.

272.—Minimum physical properties, as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch........80 000
Elongation: percentage in 2 in................... 20

273.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1½ in., without fracture on the outside of the bent portion. The bending tests may be made by pressure or by blows.

274.—Turnings from the tensile test specimens, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether the melt is within the limits in chemical composition.

Boiler Plates.

275.—The steel used for boilers and fire-boxes shall be made by the open-hearth process. Qualities of Boiler Plate Steel.

Phosphorus................0.04% maximum.
Sulphur...................0.04% maximum.

276.—The physical properties required shall be as follows:

Tensile strength desired, in pounds per square inch...60 000.

Elongation: minimum percentage in 8 in. $= \dfrac{1\,500\,000}{\text{Ultimate strength.}}$

Character of fracture.................................Silky.
Cold bends, without fracture.......................180° flat.

277.—The ultimate strength shall come within 4 000 lb. of that desired.

278.—Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese, shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of each analysis shall be furnished to the Engineer or his inspector. A check analysis shall be made from the finished material, if called for by the purchaser, in which case an excess of 25% above required limits will be allowed.

279.—Specimens for tensile and bending tests for plates shall be made by cutting coupons from the finished product, which shall have both faces rolled, and both edges milled to the usual form of the standard test specimen, $1\frac{1}{2}$ in. wide on a gauged length of at least 9 in.; or with both edges parallel.

Nickel Steel for Machine Parts.

Qualities of Nickel Steel.

280.—Nickel steel shall be made by the open-hearth process.

	Plates, shapes, and bars.	Rivets.
Phosphorus shall not exceed........	0.04%	0.04%
Sulphur	0.05%	0.04%
Nickel, not less than..............	3.00%	3.25%

281.—The physical properties required shall be as follows:

	Plates, shapes, bars, and forgings. Pounds per square inch. Minimum.	Rivets.
Tensile strength......	80 000	60 000 to 70 000
Elastic limit..........	50 000	40 000 minimum.

Elongations: percentage in 8 in., for plates, shapes, bars, and forgings; and also for rivets $= \dfrac{1\,600\,000}{\text{Ultimate strength}} =$ minimum.

Elongation: percentage in 2 in. for forgings = 25.

282.—Specimens cut from forgings (1 in. by $\frac{1}{2}$ in.) shall bend, cold, 180°, around a diameter of 1 in., without fracture on the outside of the bent portion.

283.—Specimens cut from plates, shapes, and bars shall bend, cold, 180°, around a diameter of three times their thickness, without fracture on the outside of the bent portion.

284.—Each rivet rod shall bend 180°, flat, on itself, without fracture on the outside of the bent portion.

285.—Rivet rods shall be tested as rolled.

286.—The fracture of all tension tests shall show a fine silky texture, of a uniform bluish gray or dove color, free from black or brilliant specks, and shall show no sign of crystallization.

287.—All nickel-steel forgings shall be properly annealed.

288.—Annealed eye-bars and similar members, when full-sized pieces are tested, shall comply with the following requirements:

Minimum ultimate tensile strength, in pounds per square inch..................................	75 000
Minimum elastic limit, in pounds per square inch..	45 000
Minimum elongation in 10 ft., including fracture..	12%

The fracture shall be mostly silky, and free from crystals.

Full-sized pieces shall bend, cold, 180°, around a diameter of twice their thickness, without fracture.

Tool Steel.

289.—Tool steel is generally used for parts which require hardening or oil tempering, such as pivots, friction rollers, ball-bearings, and springs. Qualities of Tool Steel.

290.—Tool steel shall be made by the open-hearth or crucible process.

Carbon	1.00%	minimum.
Phosphorus	0.04%	maximum.
Sulphur	0.04%	maximum.
Manganese	0.50%	maximum.

Phosphor-Bronze.

291.—Special phosphor-bronze shall be used for high pressure and slow speed. Qualities of Phosphor-Bronze.

292.—Phosphor-bronze shall be a copper-tin alloy. The phosphorus shall not exceed 1 per cent. Other alloys, up to one-half of 1%, will be permitted, except that no sulphur will be allowed.

Compression:

Elastic limit, in pounds per square inch	19 000	to	23 000
Permanent set, under 100 000 lb., in inches	0.12	to	0.16

293.—The compression is to be made on a cylinder having a height of 1 in. and an area of 1 sq. in. The elastic limit is to be the load which gives a permanent set of 0.001 in.

Tension:

The yield point, ultimate strength, and elongation in 2 in. are to be recorded. The tension specimen is to have a diameter of ½ in.

294.—For every heat, at least two tests shall be made. A chemical anaylsis shall be furnished.

Babbitt Metal.

295.—Babbitt metal composed of the following ingredients and of the following proportions has given satisfactory results and a low coefficient of friction (0.03 to 0.04): Qualities of Babbitt Metal.

Copper	3.6	per cent.
Tin	89.3	" "
Antimony	7.1	" "

It is the purpose of these specifications to provide a first-class structure. They are intended as an aid in designing and fabrication. Machine design and kindred subjects are so great and varied that no single work of this character can cover all points. As a further aid in secur- Purpose of the Specifications.

ing a first-class structure, the following works will be considered authoritative, in the order named:

1. Unwin's "Machine Design," Part I, Ed. 1909.
   Unwin's "Machine Design," Part II, Ed. 1902.
2. "A Manual of Machine Design," etc., by Low and Bevis, 11th Impression.
3. Reuleaux's "Constructor," Translated by Suplee.
4. Kent's "Pocket Book," 8th Ed.

296.—Machine parts shall be designed, if practicable, by the methods of applied mechanics, but such designs shall be viewed in the light of experience. It should be borne in mind that machine design is not based on the precise methods in vogue for stationary structures.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PREVENTION OF MOSQUITO BREEDING.

Discussion.*

BY THOMAS H. MEANS, M. AM. SOC. C. E.

THOMAS H. MEANS, M. AM. SOC. C. E. (by letter).—This is a timely paper and of great value. We have too long considered the mosquito a sort of necessary evil, and it is of great importance to show that it can be prevented, generally at no large expense. The writer well remembers, during boyhood, the old rain barrel, at the corner of the house, filled with wigglers. No one then connected the wiggler with the pest of mosquitoes which bothered them all summer, and, of course, no one had any idea that the chills and fever were in any way connected with the mosquitoes. Mr. Means.

In irrigated districts, mosquitoes rapidly become a great nuisance, unless special precautions are taken to eliminate their breeding places. Very little systematic work seems to have been attempted in the matter of mosquito prevention in the West. Few engineers seem to realize how easy it is to lessen the number of mosquitoes to such a degree that they cease to be a pest. It is to be hoped that Mr. Miller's paper will serve to increase the interest in the matter.

Within the last two years the writer has been engaged on a small irrigation project in the Sacramento Valley. Here, as everywhere else, mosquitoes are a nuisance, but here the matter is much complicated by the presence of the species which distributes malaria. The problem of preventing or lessening malaria is vital to the success of the project. During the summer of 1911, the first year of the writer's connection with the enterprise, no attempt was made to control the matter, largely because no one appreciated its importance. Experience during that season, however, showed the absolute necessity of sub-

* Continued from February, 1913, *Proceedings*.

Mr. Means. duing the malaria mosquito. Plans were made and work was commenced in the season of 1912.

The problem was attacked from all sides. An attempt was made to prevent the breeding of mosquitoes, to isolate the human carrier of the disease, and to encourage the people of the community to take good care of their general health by sleeping under screens, by using good drinking water, and by taking quinine as a preventive. There was no measure of the amount of malaria in the community previous to 1912, except the records kept by the school teachers. These showed that for the previous three years about 80% of the children of school age were sick with the disease. In the fall of 1912, after the one season's work, the percentage of children sick with malaria was reduced to about 15. During this year there had been a great influx of new people, and, had there been no change in the conditions, a much larger percentage of sickness might have been expected, for the disease is always worse in the case of new comers. The writer feels that he has reason to be pleased with the results.

Whether or not the season was favorable, no one can say; malaria in the country generally seemed to be about as usual in 1912. Very few farmers have much faith in the mosquito theory of malaria, and it was hoped, at least, to establish confidence in the method of handling the matter. There was very little co-operation during the year from the farmers, on account of their scepticism, but it is found that now nearly all of them are satisfied that malaria prevention is possible, that mosquitoes have something to do with its spread, and next year they will give their hearty help, and the work should be more successful.

In laying the plans, attention was directed to drainage, as the first and most important matter. Open drains were made, in order to prevent surface water from standing. Where drainage was not possible, oil was used to prevent mosquito breeding. Two men, equipped with knapsack sprays, made regular trips over the country and sprayed every place where mosquitoes could breed. These trips were made at least every two weeks in the warm weather. The territory covered was about 10 000 acres. During the season, 1 400 gal. of oil were used. The total cost of the work was about $3 500. As there were about 1 200 people in the district patrolled, the cost, if pro rated, would have been about $3 per person, which is trifling when the results are considered. The relief from mosquito bites alone would have been well worth the cost.

It has been stated that mosquitoes travel only short distances. This may be true in an unsettled country, but, where there are many people or animals, they will travel great distances. In the evening every traveler will be followed by a swarm of the insects, and every cow coming from pasture will bring them with her.

It is of importance, therefore, to exercise care in preventing the breeding of mosquitoes, even in remote spots. Mr. Means.

Malaria mosquitoes seem to prefer the quieter, warmer spots of water. They do not seem to breed in ditches or streams, but every little puddle will produce great numbers. They are a little slower in development than the common varieties, and do not become abundant until the latter part of the season.

As the result of the season's work, the writer would suggest the following for a community troubled with malaria:

1. Drain all surface water.

2. If drainage is not possible, use oil. The cheaper grades of oil from the refineries, such as stove oil, mixed with crude oil, will answer very satisfactorily. Use a fine spray, and distribute enough oil to form a thin, but visible, film on the surface of the water.

3. Keep the person, known to have malarial parasites in the blood, isolated if possible. Mosquitoes do not generate malaria, but must get it from the blood of a human being. Insist that all malarial persons sleep under screens. Have every such person undergo a thorough treatment under a good doctor.

4. Encourage the use of good water. Deep wells are generally best. Malaria cannot be transmitted through water, but every condition which tends to run the system down will make a person less resistant to the disease.

5. Screen all houses, and, as a special precaution, screen all sleeping places. Keep indoors as much as possible after night. The malarial mosquito is a nocturnal insect.

6. Take quinine during the season when malaria is prevalent. A doctor's advice should be asked. Take a little quinine, no matter what he advises.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

IRRIGATION AND RIVER CONTROL IN THE COLORADO RIVER DELTA.

Discussion.*

By Messrs. Francis L. Sellew and Elwood Mead.

Francis L. Sellew, M. Am. Soc. C. E. (by letter).—Turning the Colorado out of Salton Sea was not so much an engineering triumph as a triumph of engineers, of red-blooded fighters, who did not know when they were whipped, but realizing that the end justified the means, fought on to victory unmindful of its cost. All honor to them. The writer will go to the limit in expressing appreciation of results attained in the face of odds as great as any with which river engineers may expect to be confronted; but when the closing of this crevasse, which, from the nature of things, had to be accomplished by rough and ready methods, is heralded as a solution of all engineering problems pertaining to the Lower Colorado, the writer's protest is immediately entered. Mr. Sellew.

Having been an innocent bystander while some of the work described was in progress, this paper has been read with much interest. The record of the operations incident to the return of the river to its former channel is extremely complete, although partly obscured by a mass of legal and financial entanglements which properly have no place in a purely engineering article.

Lack of time will confine the writer's discussion to an examination of some of the more important points. The data submitted in regard to the discharge of the stream and the degree of reservoir control possible vary materially from the writer's ideas on these subjects. It is also doubtful if the control attainable by reservoirs will greatly reduce bank caving, which is the real problem along this river. While what

*Continued from February, 1913, *Proceedings*.

Mr. Sellew. follows is based on the best data in existence, it should be understood that, before definite conclusions can be drawn, much more detailed and extended examinations should be made.

Flood Prevention on the Lower Colorado.

It is apparent that if the flood discharge of the Colorado could be reduced materially, thereby bringing the fluctuations between low and high water within reasonable limits, the erosion of the bed and banks would be much reduced. There might still be a limited amount of bank protection required, but, if the control was such as to keep the river within banks, the cost of protective works would be greatly lessened. We have recently heard much about the control of streams by storage works; and, with reference to the Colorado, one advocate claims that storage near the head-waters will reduce flood levels so much that levees will be a thing of the past; that the flow of the stream may be entirely devoted to irrigation, thereby putting water on 5 000 000 acres of land; and also that immense quantities of electrical energy may be developed as the water is released from the places in which it is impounded. As the question of stream control by storage reservoirs has received much attention in most civilized portions of the earth, the writer has made a somewhat extended examination of the literature on that subject.

On page 406 of the "Report upon the Physics and Hydraulics of the Mississippi River," by Humphreys and Abbot, the following appears, in regard to the reservoir system:

"The plan, in theory, is admirable, and has long been a subject of discussion among European engineers. Artificial lakes for protection against floods were constructed as early as 1711 upon the upper Loire, and they have since been advocated, both for improving navigation and for restraining floods, by eminent writers. * * *."

From page 407 of the same publication:

"Little consideration is necessary to make it apparent that this system is not applicable to restraining the floods of all rivers. Certain topographical conditions are essential to its success. The valley must be of such a character that dams of reasonable dimensions can be constructed, which shall keep back *the identical water which otherwise would make up the flood.* It is not sufficient for this purpose, as for improving navigation, that a large volume of water may be collected by the accumulations of months. The floods of great rivers are torrents, caused by rapidly melting snows and by widely extended and heavy rains. The greater part of this water does not drain from the remote mountain sides, and issue from the distant mountain gorges. It falls in the valley itself; and the nearer to the main river, the more sudden and disastrous will be its effects; partly from the more rapid accumulation in the main stream of the contributions of the tributaries, and partly from the absence of the natural reservoir furnished by the various channels, which must be filled before a freshet originating near the sources can reach the lower part of a river. To control

such floods with certainty and economy by artificial reservoirs, it is, therefore, essential that certain important tributaries which drain relatively large portions of the basin shall debouch near their mouths from narrower gorges, where dams can be constructed at reasonable cost, and where artificial lakes can be formed without injury to other interests." Mr. Sellew.

From page 408:

"In order to give a more definite character to these conclusions, they will be reduced to figures by aid of the data collected respecting the great June flood of 1858, by which the merits of all these different plans of protection are to be tested.

"To have protected 'the whole delta and the borders of every stream in it, primary or tributary' against this flood, not more than 1 050 000 cubic feet per second could have been allowed to enter the head of the alluvial region. Even this quantity would have submerged much of the lower country, had not the tributaries below the Ohio been so very low that their united contributions, joined to this amount, would only have been sufficient to maintain the river at full banks. The conditions of this flood were then the most favorable possible for the reservoir system.

"During the thirty-six days in 1858 from May 25 to June 29, inclusive, the total amount of water passing the latitude of Columbus exceeded by 648 172 800 000 cubic feet that which would have resulted from a discharge per second of 1 050 000 cubic feet. Reservoirs situated above the mouth of the Ohio, and sufficient to have kept back *in a single month* fully 600 000 000 000 cubic feet of water [14 000 000 acre-ft.] would, therefore, have been essential to the security of the delta, if this system had been depended upon for restraining this flood."

From a report on the examination of reservoir sites in Wyoming and Colorado, by Hiram M. Chittenden, M. Am. Soc. C. E., Captain (now General, Retired), Corps of Engineers, U. S. A.:*

"In no other portion of her works has nature left so much to be done by the engineer to supplement her deficiencies as in the modification of the natural flow of her streams, for in no other respect are her works so ill adapted to the uses of man. The ideal stream would be one in which the flow should be uniform from one year's end to the other, or, if not uniform, varying directly with the magnitude of the uses to which it is put.

* * * * * * *

"It is not surprising, therefore, that one of the chief concerns of the engineer is the amelioration or prevention of the evils of this unfortunate arrangement of nature. Millions of dollars are annually expended to make up for the deficiency of water in seasons of drought, and like sums to prevent or alleviate the evils of excessive flow. Singularly enough, the measures generally adopted are put forward in disregard of one of the commonest rules of scientific practice. If an evil condition of things is to be corrected, the rational method of procedure is to remove the cause. In all river engineering, however, the measures adopted look only to the palliation of results, and leave

* Annual Report, Chief of Engineers, U. S. Army, for 1898, Appendix PP, p. 2845.

Mr. Sellew. the cause untouched. River channels are dredged out in low water and levees are built to protect from floods in high water. Scarcely anywhere is the effort made to prevent the occurrence of either high or low water. It would naturally follow that, if great evils result from the variable flow of streams, the primary and fundamental object of the engineer who is called upon to correct them would be to make this flow uniform. Whether or not this object is possible of realization (and if it is, by what means) is therefore one of the first questions which should be settled in any comprehensive project for the regulation of the flow of streams.

* * * * *

"* * * The only possible method by which uniformity of flow can be secured must therefore be by storing the surplus waters in seasons of flood and releasing them in seasons of drought.

"There is an additional motive for the use of reservoirs besides that of securing uniformity of flow. Over a large portion of the land area of the earth in civilized countries the climate of winter prevents any considerable use of the streams. Even if the flow were entirely uniform, that portion which takes place in the season of cold weather would mostly be lost. To derive any benefit from it, it must be stored and held over for the season of warm weather.

"These two purposes, viz., the attainment of uniformity of stream flow and the transfer of the winter supply to the summer months cover the entire argument for reservoir construction.

* * * * *

"At first thought it would seem that in storage reservoirs lies the whole solution of the river problem. To store the surplus in flood season and use it in season of drought ought apparently to strike at the root of the whole difficulty, and to render unnecessary those palliative measures which alone have hitherto received the sanction of the hydraulic engineer. Why so obvious a remedy has never yet been extensively applied will appear in the course of this report.

* * * * *

"It is the *cost*, not the physical difficulties, which stands in the way. It may be stated that as a general rule a sufficient amount of storage can be artificially created in the valley of any stream to rob its floods of their destructive character; but it is equally true that the benefits to be gained will not ordinarily justify the cost.*

* * * * *

"We now come to the specific question of reservoir construction in the arid region west of the one hundredth meridian, as exemplified by the reservoir sites examined in Wyoming and Colorado. Are there direct and primary motives which would justify reservoir construction in this country, apart from or in addition to those arising from their effect upon the regimen of the lower rivers? The answer must be that in no other part of the United States, nor anywhere else in the world, are there such potent and conclusive reasons, of a public as well as a private nature, for the construction of a comprehensive reservoir system as in the region here in question.†

* * * * *

* *Loc. cit.*, p. 2860.

† *Loc. cit.*, p. 2864.

"* * * in many sections the natural flow has been used as far as it is practicable to do so. The only resource left is to store that portion of the flow that runs away in nonirrigation seasons and the surplus in times of annual flood and sudden freshets and make these also available for use. Not until that is done can a stream be said to be really utilized to the fullest extent. Mr. Sellew

* * * * *

"While there are these clear and positive arguments in favor of the storage of the surplus flow of our Western streams, there are none of weight against it. It may be set down as a rule, to which there are very few exceptions, that every artificial body of water created in the West, by which the surplus water of its streams is held back, will be a positive benefit.

* * * * *

"The foregoing examination has led up to the following conclusions:

"First. A comprehensive reservoir system in the arid regions of the United States is absolutely essential to the future welfare of this portion of the national domain.

"Second. It is not possible to secure the best development of such a system except through the agency of the General Government.*

* * * * *

"The total extent of a reservoir system in the arid regions which shall render available the entire flow of the streams will not exceed 1 161 600 000 000 cubic feet [approximately 27 000 000 acre-ft.]. If the construction of such a system were to consume a century in time, it would represent an annual storage of about 11 600 000 000 cubic feet, or 266 300 acre-feet. At $5.37 per acre-foot this would cost $1 430 031 per annum. This amount, distributed among the seventeen States and Territories of the arid region, gives an average annual expenditure in each of $84 119. The annual value of the stored water would return the original cost and maintenance in an average period of three years."†

In a paper on "Reservoir Systems and Their Relations to Flood Protection,"‡ by C. O. Sherrill, Captain, Corps of Engineers, U. S. A., presented at a special meeting of the Louisiana Engineering Society on June 25th, 1912, the following appears:

"The most vital question of the moment for the people of the lower Mississippi Valley is how best to secure protection from disastrous floods, such as the one now passing, and in the proper solution of this question the sympathy and assistance of the entire country should be ready to aid. Every flood brings forth a multitude of plans, each purporting to be the only one capable of providing the necessary cure, and most of them are brought out as something entirely new, yet each one will, on examination, be found to have been carefully considered and thoroughly investigated years ago. One of these propositions, renewed recently with great energy, has been to control these floods by means of reservoirs located near the headwaters of the tributaries.

* *Loc. cit.*, p. 2872.

† *Loc. cit.*, p. 2878.

‡ *Journal*, Association of Engineering Societies, September, 1912.

Mr. Sellew. "In view of the fact that adequate reservoir systems for the control of floods in all streams would strike directly at the seat of the trouble, it seems remarkable that this method should not have been used long ago instead of the merely defensive method of elevation of overflow land or the erection of levees.

* * * * *

"* * * The question, therefore, is, Can this stream flow be made uniform; if so, how and at what cost? * * * I must answer that a reasonable degree of uniformity of flow can be secured by adequate systems of reservoirs properly located along streams where the topography is particularly adapted to such reservoirs; but as to the possibility of such control for the Mississippi below Cairo, or of the practicability of the scheme, if possible, it is hoped that the following remarks will be of some assistance in determining."

This paper concludes:

"Taking the above brief summary of the facts into consideration, I must conclude that the control of the Mississippi floods by reservoirs is impracticable of accomplishment and that the next best thing must be relied upon, namely, the levee system with bank protection, which should be completed as rapidly as possible."

The Pittsburgh Flood Commission, in a comprehensive and voluminous report covering a recent investigation, concluded that the maximum flood crests of the Ohio at Pittsburgh can be lowered approximately 8 ft. by the storage of about 1 000 000 acre-ft. of water, in seventeen reservoirs on the Allegheny and Monongahela, at a total cost of about $20 000 000.

From the foregoing it appears that the desirability and practicability of reservoir construction depend on existing conditions in the drainage area of the stream under discussion; therefore, each stream should be subject to careful analysis before definite recommendations can be made. As the Colorado receives almost its entire supply from melting snows at the head-waters, it would appear to be an ideal stream for reservoir control.

Existing Storage Works.—Before discussing the feasibility of reservoir control on the Colorado, attention is called to Tables 23 to 27, which show what has been accomplished by storage systems in various countries, with their extent and approximate cost.

Table 23 contains data on ten projects of the Reclamation Service having a total capacity of about 3 800 000 acre-ft., which have been constructed at an average cost of $2.65 per acre-ft.

Table 24 shows other existing reservoirs in America with an aggregate storage of 2 383 500 acre-ft., built at a total cost of $2 629 800, or an average of $1.10 per acre-ft. In this list the storage at the head-waters of the Mississippi was by utilizing natural lakes, the outlets of which were controlled by low, inexpensive dams, the average cost being only 36 cents per acre-ft. If this system be excluded, it

Mr. Sellew.

TABLE. 23.—Reservoirs in the United States. Built by the Reclamation Service.

Location.	Type of controlling works.	Storage, in acre-feet.	Cost. Total.	Cost. Per acre-foot.	Source of information.
Arizona:					
Roosevelt Dam	Masonry......280 ft.	1 284 000	$3 697 000	$2.90	10th Annl. Rept., U. S. R. S.
California:					
East Park	Concrete......139 ft.	45 600	239 000	5.25	" " " "
Idaho:					
Deerflat	Earth......40 to 70 ft.	186 000	868 800	4.65	" " " "
New Mexico:					
Carlsbad	Earth and rock......50 ft.	47 000	220 700	4.70	" " " "
Oregon:					
Cold Springs	Earth......98 ft.	50 000	442 600	8.85	" " " "
Klamath	Rock fill......33 ft.	462 000 *	130 000	0.28	" " " "
South Dakota:					
Belle Fourche	Earth......115 ft.	209 700	1 123 900	5.40	" " " "
Washington:					
Bumping Lake	Earth......45 ft.	34 000	410 700	12.10	" " " "
Wyoming:					
Pathfinder	Masonry......218 ft.	1 025 000 +	1 693 000	1.65	" " " "
Shoshone	Concrete......328 ft.	456 600	1 179 300	2.60	" " " "
Totals		3 799 300	$10 005 000	$2.63	

* Lake storage.
+ Drainage area, 12 000 sq. miles.

Mr. Sellew.

TABLE 24.—(*Continued*).

Location.	Type of controlling works.	Storage, in acre-feet.	Cost.		Source of information.
			Total.	Per acre-foot.	
MISCELLANEOUS RESERVOIRS.					
Spain:					
Villar	Masonry	13 050	$ 390 000	$29.90	Buckley.
Belgium:					
Gilleppe	Masonry	9 730	874 000	90.	"
Wales:					
Vyrnwy	Masonry	44 690	3 334 000	75.	"
Australia:					
Beetaloo	Concrete	2 945	573 300	195.	"
Totals		70 415	$5 171 300	$74.	

Mr. Sellew.

TABLE 24.—VARIOUS RESERVOIRS.

Location.	Type of controlling works.	Storage, in acre-feet.	Cost. Total.	Cost. Per acre-foot.	Source of information.
IN THE UNITED STATES.					
Mississippi Reservoir System.	Earth and Timber Dams....	2 100 000	$ 750 000	$0.36	Pittsburgh Flood Commission.
Texas:					
Pecos River..................					
Upper Reservoir.............	Earth......................	82 640	170 000	2.06	Chief of Engineers, 1898, Ap. PP, p. 2832.
Lower Reservoir.............	Earth......................	7 000	86 000	12.29	Chittenden Report.
Colorado:					
Larimer and Weld	Earth......................	7 460	90 000	12.00	" " p. 2842
Cache la Poudre	Earth......................	5 650	125 000	22.20	" " " "
California:					
Escondido....................	Rock fill....................	3 500	110 000	31.44	" " " "
Cuyamaca....................	Earth......................	114 755	959 800	8.37	" " " "
Sweetwater..................	Masonry....................	22 500	264 000	11.70	" " " "
Bear Valley..................	Masonry....................	40 000	75 000	1.88	" " " "
Totals.................		2 383 505	$2 629 800	$1.10	
SUGGESTED RESERVOIRS—GEN. CHITTENDEN.					
Wyoming:					
Laramie......................	Masonry ....80 ft.	414 000	$1 416 000	$3.42	Chittenden Report, p. 2842.
Sweetwater	Masonry..............100 ft.	326 900	276 800	0.85	" " " "
Piney Creek..................	Masonry.........3 low dams	85 400	214 600	2.52	" " " "
Colorado:					
South Platte	Masonry..............200 ft.	67 200	540 000	8.04	" " " "
Loveland.....................	Masonry....................	45 700	262 100	5.73	" " " "
Totals..................		939 200	$2 709 500	$2.90	

Mr. Sellew. appears that the remainder amounts to only 283 500 acre-ft., which cost about \$6.65 per acre-ft. In this table there are also five sites in Wyoming and Colorado, suggested by Gen. Chittenden, aggregating 939 200 acre-ft., estimated to cost \$2 709 500, or an average of about \$2.90 per acre-ft. This table also shows 70 415 acre-ft. of storage in Europe and Australia, which have cost \$74 per acre-ft.

Table 25 contains data on ten works in France, having a total capacity 38 593 acre-ft. and costing \$3 518 900, or about \$91 per acre-ft. This table also shows that 38 400 acre-ft. of storage in Austria has cost \$232 per acre-ft., while in Canada reservoirs containing 3 800 000 acre-ft. are being created on numerous lakes on the Ottawa River at an estimated cost of 20 cents per acre-ft.; and further, that about 700 000 acre-ft. in South Africa have been built at an average cost of \$14 per acre-ft.

Table 26 gives a list of eleven projects in Germany, volume 488 111 acre-ft., cost \$22 449 874, or \$46 per acre-ft.; shows that Russia has 800 000 acre-ft. on the Volga (no records of cost); and that five systems in India, aggregating 775 250 acre-ft., cost \$6.77 per acre-ft. The storage of about 1 900 000 acre-ft. at the Assuan Dam, in Egypt, has cost about \$10 per acre-ft.*

The completed storage (Canada being excluded as under construction) is summed up in Table 27.

Although there are many storage dams for water supply and power in various parts of the world, they are of such relatively low capacity and high unit cost that they have no place in a discussion of this kind. The foregoing data are not submitted as complete, but include all the larger systems for river control, of which records are available to the writer; and it is believed that they cover the field quite thoroughly.

It appears that the storage in the United States (which is about 50% greater than that of the rest of the world) as listed in these tables, has cost on the average only one-ninth as much per unit of volume, while the maximum cost, that of Austria, is about 116 times the average in America. The low cost of the storage created by the Reclamation Service, when compared with that of the rest of the world, is shown in a most remarkable manner.

Determining Factors.—The feasibility of controlling a stream by storage will depend largely on:

(1) The cost, including the value of the area flooded, when compared with the benefits to be derived;

(2) The quantity of storage required and whether sufficient volume is available.

* *Engineering Record*, January 11th, 1913.

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TABLE 25.—Reservoirs in France, Austria, Canada and South Africa.

Location.	Type of controlling works.	Storage. in acre-feet.	Cost. Total.	Cost. Per acre-foot.	Source of information.
Reservoirs in France.					
Rhone River	Recommended as too expensive for reservoir control				Pittsburgh Flood Commission.
Garonne River					" " "
Loire River	Recommended for construction, but not built because of expense				" " "
Seine River	Reservoir control not applicable. Scarcity of sites				" " "
Furens Dam	Masonry, 181 ft	1 300	$318 000	$244	" " "
Ternay Dam	Masonry, 119 ft	2 100	204 372	97	" " "
Curzon	Masonry	1 297	247 600	190	Buckley.
Ban	Masonry	1 499	190 000	127	"
Pas du Riot	Masonry	1 054	256 000	243	"
Chartrain	Masonry	3 647	420 000	115	"
Lake Oredon	Earth	5 891	142 000	24	"
Mouche	Masonry	7 011	1 003 657	143	"
Liez	Earth	13 051	598 418	46	"
Wassy	Earth	1 740	138 942	80	"
Totals		38 593	$3 518 989	$91±	
Reserviors in Austria.					
Oder River	Six Reservoirs	5 100*	$1 488 000	$292	Pittsburgh Flood Commission.
Elbe and Tributaries	Four Projects proposed	13 000	2 067 000	159	" " "
Wien River	Six Projects	19 000	3 661 000	193	" " "
	One Project	1 300	1 680 000	1 292	" " "
Totals		38 400	$8 896 000	$232±	

* Drainage area, 29 sq. miles.

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TABLE 25.—(*Continued*).

Location.	Type of controlling works.	Storage, in acre-feet.	Cost.		Source of information.
			Total.	Per acre-foot.	
RESERVOIRS IN CANADA.					
Ottawa River	Three projects. Low concrete dams on numerous lakes	3 800 000	$728 000	$0.20	Pittsburgh Flood Commission.
RESERVOIRS IN SOUTH AFRICA.					
Cape Colony	Earth, six dams	55 422	$1 700 000	$30.80	Buckley.
	Concrete, five dams	44 274	2 800 000	63.20	"
Transvaal	Earth, one dam	95 000	684 000	7.20	"
	Concrete and weirs, four dams	505 000	4 646 000	9.20	"
Totals		699 696	$9 830 000	$14.00	

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TABLE 26.—Reservoirs in Germany, Russia, and India.

Location.	Type of controlling works.	Storage, in acre-feet.	Drainage area, in square miles.	Cost. Total.	Cost. Per acre-foot.	Source of information.
Reservoirs in Germany.						
Wupper River:						
Bever Valley	Masonry.....52.5 ft.	2 700	9	$343 200	$127	Pittsburgh Flood Commission.
Lingese Valley	Masonry.....61 ft.	2 100	4	256 800	121	" " "
Ruhr River:						
12 Reservoirs	Masonry.....64 to 114 ft.	33 000	71	3 480 000	105	" " "
Rur River:						
Urft Reservoir	Masonry.....190 ft.	37 000	145	1 000 000	27	" " "
Weiseritz River:						
Malter Reservoir	Masonry.....115 ft.	7 000	40	883 000	126	" " "
Kleingenburg Reservoir	Masonry.....128 ft.	12 000	35	858 000	72	" " "
Weser River	Masonry.....136 ft.	164 000	552	4 500 000	27	" " "
Oder River:						
Glatzer Neisse, Proposed	Earth.....37 ft.	82 500	906	3 840 000	47	" " "
Malapane Reservoir	Earth.....33 ft.	72 000	403	2 880 000	40	" " "
Tributaries in Silesia	13 Reservoirs completed or under construction	75 000	.....	4 317 000	58	" " "
Remscheid	Masonry	811	.....	91 154	112	Buckley.
Totals	.....	488 111	.....	$22 449 874	$46	
Reservoirs in Russia.						
Volga Reservoirs	Low dams on numerous lakes.	800 000	.....	.....	.....	Pittsburgh Flood Commission.
Reservoirs in India.						
Tansa	Masonry	52 670	.....	$988 000	$18.76	Buckley.
Betwa	Masonry	36 800	.....	160 000	4.35	"
Chumbrumbaukum	Earth	63 780	.....	312 000	4.89	"
Bombay:						
20 tanks	Earth	372 000	2 500±	2 090 100	5.62	"
4 tanks	Masonry	250 000	450	1 700 000	6.80	"
Totals	.....	775 250	.....	$5 250 100	$6.77	

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(3) The situation of such storage, whether it is placed so as to control properly the water-producing zone of the drainage area;

(4) The character of the water to be impounded, whether it is clear or so charged with sediment that the silting will unduly shorten the life of the works.

TABLE 27.—SUMMARY OF COMPLETED RESERVOIRS.

	Acre-feet.	Cost.
FOREIGN.		
Egypt	1 900 000	$19 750 000
France	38 593	3 518 000
Germany	488 111	22 498 000
Austria	38 400	8 896 000
Russia—no cost recorded	....	....
India	775 250	5 250 000
South Africa	699 696	9 830 000
Miscellaneous foreign	70 415	5 171 000
	4 010 465	$74 913 000

$\frac{74\ 913\ 000}{4\ 010\ 465}$ = $18.70 per acre-ft.

	Acre-feet.	Cost.
UNITED STATES.		
Reclamation Service	3 799 300	$10 005 000
Other systems	2 383 500	2 629 800
	6 182 800	$12 634 800

$\frac{12\ 634\ 800}{6\ 182\ 800}$ = say, $2.00 per acre-ft.

These factors, as applied to the Colorado, will be considered in the foregoing order.

The Cost.—From what has already been written, it appears that, where storage exists in the arid regions, its cost is reasonable. Assume that the cheapest sites have been taken up and that future storage will cost twice as much as that already created by the Reclamation Service ($2.65 per acre-ft.), and we have $5.30 per acre-ft. This is in close agreement with Gen. Chittenden, who, as previously stated, estimated $5.35 for all storage needed in the Arid West. Of course, each site must be carefully examined, and detailed estimates must be prepared, which will no doubt vary considerably from the foregoing, but, from what has been stated, the figure given appears to be a fair and reasonable one to use in this discussion.

Volume of Storage Required.—An intelligent discussion of this phase of the matter would be greatly aided by continuous discharge

measurements on the Colorado River at various points, and extending over a long period. Unfortunately, however, such measurements do not exist. The most complete records are those made at Yuma since November, 1902, by the Reclamation Service. These observations include current meter measurements three times each week. Previous records of the flow at Yuma are based on such insufficient data—being a few scattered meter measurements from which a rating-curve was constructed—that they are of no value in this discussion. The wide variation in the quantity flowing for the same gauge height, due to the change in cross-section caused by scour, renders observations without a meter measurement practically worthless. Mr. Sellew.

Table 28 shows the total discharge of the Colorado (excluding the Gila) by months, in acre-feet, the mean monthly discharge in second-feet, and also the total discharge in acre-feet per year from 1903 to 1912, inclusive. This table shows that the yearly discharge has varied from a minimum of 9 800 000 acre-ft. in 1904, to a maximum of 25 300 000 acre-ft. in 1909. The total monthly discharge has varied from a minimum of 182 500 acre-ft. in February, 1903, to a maximum of 6 397 000 acre-ft. in June, 1912.

In considering the uses of the stream for irrigation, it is proper to forecast as nearly as possible the lowest flow that is to be anticipated. In doing this, the writer has compiled Table 29, showing what may be called an "ideal minimum year", and also the monthly use of water for irrigation at Yuma. This table has been constructed by selecting from Table 28 the minimum January discharge, the minimum February discharge, etc., for all months included in the latter table. This gives a minimum year containing a total discharge of 8 315 500 acre-ft., and these discharges are shown in monthly totals on the diagram, Fig. 36.

For the present, the duty of water in this vicinity is assumed as $5\frac{1}{2}$ acre-ft., its use being distributed throughout the year as shown in Table 29. It is hoped, and indeed expected, that this use of water will be reduced considerably as larger areas are put under cultivation and the settlers become more skilled in the application of water, but for the present this appears to be the best distribution for calculations, in view of the fact that the use now is considerably in excess of $5\frac{1}{2}$ acre-ft. On Fig. 36 is also platted the use which can be made of the stream for irrigation without any regulation whatever. This was ascertained in the following manner: The use of water in February being about 0.25 acre-ft. and the lowest run-off in that month being 182 500 acre-ft., it was found that this quantity was sufficient for the watering of 730 000 acres. This was assumed as the limiting month. If greater areas were placed under cultivation, there would be a shortage in that month at a time when the new crops are usually going in. Taking this 730 000 acres as a basis, and platting the monthly duty

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TABLE 28.—DISCHARGE OF COLORADO RIVER AT LAGUNA DAM IN TOTAL ACRE-FEET PER MONTH AND MEAN SECOND-FEET.

Year.	January.		February.		March.		April.		May.		June.	
	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.
1903	185 407	3 015	182 531	3 286	367 722	5 980	788 957	13 268	1 957 151	31 830	3 116 946	52 382
1904	223 630	3 637	217 813	3 787	367 702	5 980	479 663	8 061	1 699 554	27 641	2 599 533	43 687
1905	310 680	5 053	879 894	15 843	2 087 202	33 945	1 482 419	24 913	2 293 324	37 298	4 507 351	75 748
1906	286 880	4 649	362 795	6 532	986 040	16 036	1 512 098	25 412	3 201 350	52 065	5 005 070	84 113
1907	1 256 500	20 470	980 600	17 730	1 191 000	19 410	2 028 500	34 140	2 330 000	37 900	5 640 000	94 800
1908	389 000	6 320	425 500	7 400	827 353	13 590	1 060 000	17 800	1 670 000	27 200	2 550 000	42 900
1909	543 732	8 800	597 629	10 700	827 715	13 500	1 708 981	28 687	3 310 561	53 840	6 210 488	104 873
1910	945 733	15 335	499 346	9 100	1 498 485	24 400	1 708 438	28 700	3 472 513	56 500	2 798 123	47 000
1911	541 487	8 800	742 610	13 400	1 067 700	17 400	1 213 685	20 400	2 764 960	45 000	3 818 576	64 200
1912	331 244	5 400	423 676	7 370	697 002	11 770	1 183 552	19 890	2 507 932	40 800	6 397 382	107 510

Year.	July.		August.		September.		October.		November.		December.		The Year.	
	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.	Total acre-ft.	Mean sec-ft.
1903	2 228 065	36 735	589 166	9 581	398 736	6 701	504 869	8 211	321 271	5 399	266 598	4 336	10 907 419	15 060
1904	1 406 203	22 870	914 548	14 874	649 802	10 920	689 185	11 108	359 523	6 041	275 305	4 477	9 882 461	13 590
1905	1 859 380	30 240	744 111	12 102	383 496	6 445	482 980	7 855	440 770	7 445	571 720	9 298	16 043 327	22 180
1906	2 395 311	38 956	1 148 779	18 683	694 300	11 668	720 020	11 710	575 603	9 673	534 235	8 680	17 422 481	24 020
1907	5 930 000	96 400	2 309 600	37 600	1 286 800	21 640	778 000	12 640	629 400	10 570	458 000	7 450	24 818 400	38 230
1908	2 000 000	32 600	1 395 288	22 800	633 818	10 700	585 000	9 510	481 000	8 090	573 961	9 300	12 590 920	17 340
1909	4 875 840	79 297	2 453 986	39 910	2 807 545	47 138	860 839	14 000	561 917	9 400	517 098	8 400	25 306 334	34 880
1910	904 476	14 700	591 480	9 600	366 948	6 200	429 428	7 000	466 916	7 800	426 770	6 940	14 108 656	19 440
1911	3 048 838	49 450	1 131 983	18 400	530 388	8 900	1 726 538	28 100	705 135	11 850	465 130	7 600	17 757 030	24 160
1912	2 854 852	46 430	1 356 912	22 073	581 959	9 780	675 382	10 990	699 184	11 750	403 444	6 560	18 357 686	25 360

in the ratio shown in Table 29, it appears that the volume of water used will be that shown by the shaded area in Fig. 36. This arrangement may create a slight shortage in September, but the use in that month is so near the minimum run-off that the danger is considered of slight importance and is neglected. Mr. Sellew.

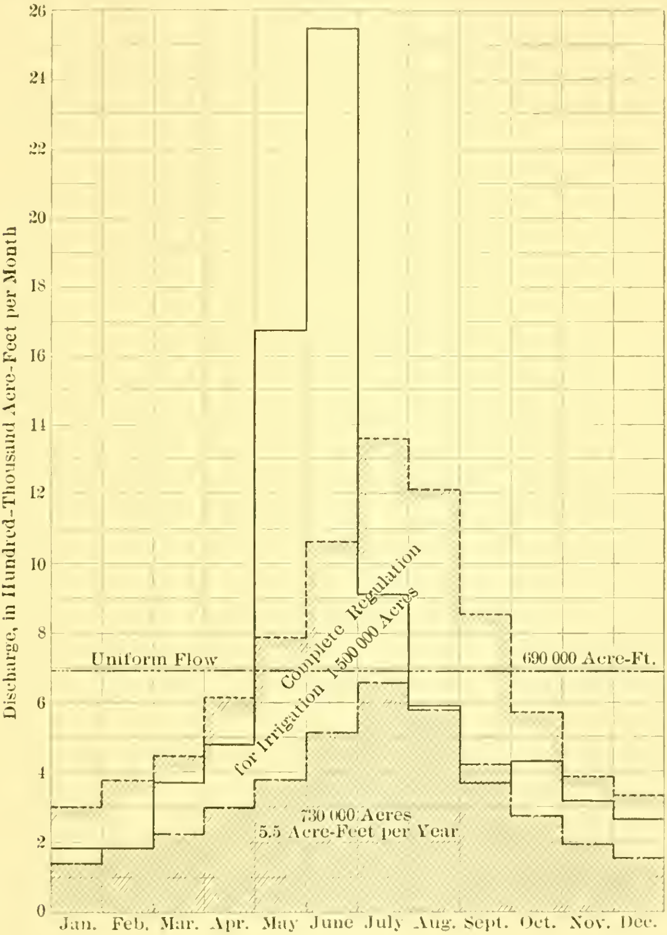


FIG. 36.

It is well to remark that the continuous irrigation of 730 000 acres depends on the diversion of the entire river, and is substantially all that can be accomplished, unless the stream is regulated, even should the duty of water be increased, for it is seen that the use of water at the extremes of the year is as low as can be anticipated, and

Mr. Sellew. although a saving may be made in the summer months, it will allow an increase in the cultivated area only during those months, because the crops which can be grown continuously for twelve months in the year are limited by the small quantity of water available during low flow.

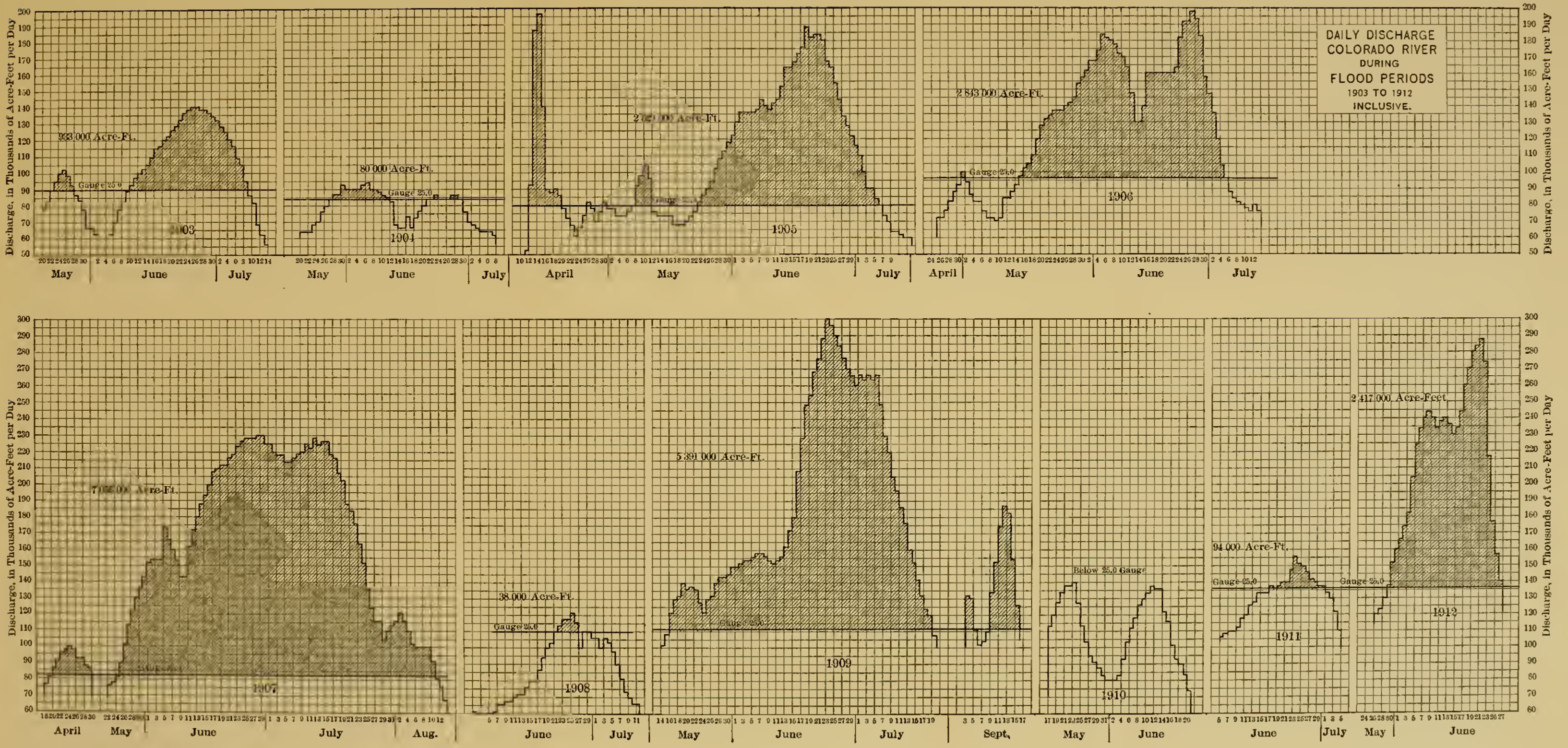
TABLE 29.—MINIMUM MONTHLY DISCHARGE OF COLORADO RIVER AND DISCHARGE IN RATIO OF MONTHLY DUTY.

Month.	Minimum discharge, in acre-feet.	Monthly duty.	Percentage of total yearly duty.	Monthly discharge for year in ratio of duty.
January	185 407	0.20	3.6	302 801
February	182 531	0.25	4.5	378 000
March	367 702	0.30	5.4	453 500
April	479 663	0.41	7.4	619 801
May	1 670 000	0.52	9.4	786 100
June	2 550 000	0.70	12.8	1 058 100
July	904 476	0.90	16.4	1 360 700
August	591 480	0.80	14.6	1 209 500
September	363 948	0.56	10.3	846 701
October	429 428	0.38	6.9	574 500
November	321 271	0.26	4.7	393 150
December	266 598	0.22	4.0	332 651
Totals	8 315 594	5.50	100.0	8 315 504

There is drawn on Fig. 36 a line showing the uniform flow throughout the year, which would be at the rate of 690 000 acre-ft. per month. The establishment of such uniformity in discharge would be of value in controlling the floods and also for power purposes, but it is extremely wasteful toward the irrigation interests, because this line is practically coincident with the maximum use of water in July for 730 000 acres. The reinforcement of the low flow by this method would not allow an increase in the area under crop in the summer. It would cover more ground, however, during the winter.

Table 29 also shows the percentages of irrigating water required per month, and on this basis the discharge of the low-water year of 8 315 500 acre-ft. has been distributed. This assumes complete regulation of the river, entirely in the interests of irrigation, and the distribution throughout the year is shown by the shaded line. Under such regulation, 1 500 000 acres (which agrees with the author's estimate) could be irrigated. This is the area that could be served with the flow of the Colorado past Yuma as observed during the years under discussion. The lower Gila is so erratic in its discharge that it is not a very dependable supply, for which reason it is omitted. To the foregoing should be added any area existing in the upper reaches already under cultivation and which it is assumed has received its supply. To regulate the river as indicated would require a storage capacity of about 2 400 000 acre-ft., which, at $5.30 per acre-ft.,

PLATE XLIX.
PAPERS, AM. SOC. C. E.
MARCH, 1913.
SELLEW ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



would cost $12 720 000. Such regulation in low-water years would also have a very beneficial effect on the flood discharge. The maximum monthly volume would be about 1 400 000 acre-ft., equivalent to approximately 50 000 acre-ft. per day, or 25 000 sec-ft. The effect of this discharge on the bed and banks would be but slight, as compared with what goes on at the present time, and would keep the lower river well within its banks. Mr Sellew.

Although this amount of storage would make the river an ideal one for irrigation during periods of ordinary low flow, it would have little beneficial effect on floods in the maximum years, as will be seen from an inspection of the diagrams on Plate XLIX. Here the daily discharge in acre-feet during the flood periods is shown for the years 1903 to 1912, inclusive.

When the river at Yuma rises above 125 on the local gauge, overflow occurs at various points within the limits of the Yuma Project, and levees become a necessity. If the control of the stream by storage is to regulate the discharge in such a way that levees will not be required, the volume passing when the gauge is above 125 must be stored. On the diagrams referred to a horizontal line has been drawn representing 125 on the Yuma gauge, and the volumes of discharge above this gauge height in various years are clearly marked. They were:

In 1903	933 000	acre-ft.
1904	80 000	" "
1905	2 829 000	" "
1906	2 843 000	" "
1907	7 055 000	" "
1908	38 000	" "
1909	5 391 000	" "
1910 (the gauge did not rise above 125)		
1911	94 000	" "
And in the season just passed.............	2 417 000	" "
A total of 21 680 000 acre-ft., or an average of	2 168 000	" "
for the ten-year period.		

If storage is to be created for the maximum of these quantities, 7 055 000 acre-ft.—and this is necessary if all overflow is to be prohibited along the lower river—the cost of storage, if sufficient sites existed, would be $37 391 500, estimated at $5.30 per acre-ft. Should this be considered an excessive provision to make, and it were thought better to take what damage would come from a flood of this kind (which might be of infrequent occurrence), and storage were provided for the next largest year, which is 1909, 5 391 000 acre-ft. would be required, which would cost, on the same basis, $28 572 300. This would give a protection that would have been ample during the last 10 years, with one exception. If we take the average of the excess

Mr. Sellew. discharge at Yuma above the 125 gauge for the last 10 years, we get 2 168 000 acre-ft., which is about that required to regulate the low flow in the interests of irrigation. The cost of creating this amount of storage at $5.30 would be $11 490 400. With such control of the river, however, overflow would have occurred in 5 years during the last 10.

Attention is directed to Table 3, which is an exhibit of the annual discharge of the river from 1894 to 1911, inclusive, a period of 18 years. The first 9 years have an average discharge of 7 220 000 acre-ft., and the last 9 years show an average of 17 556 000 acre-ft., indicating a most remarkable, sudden, and continued increase in run-off. Such an increase might be possible with a small drainage, but, with a tributary area of more than 250 000 sq. miles, the variation appears to be altogether too much. To aid in the examination of this question, various publications of the Weather Bureau have been used in the preparation of Fig. 37. For the northern part of the water-shed, no continuous ob-

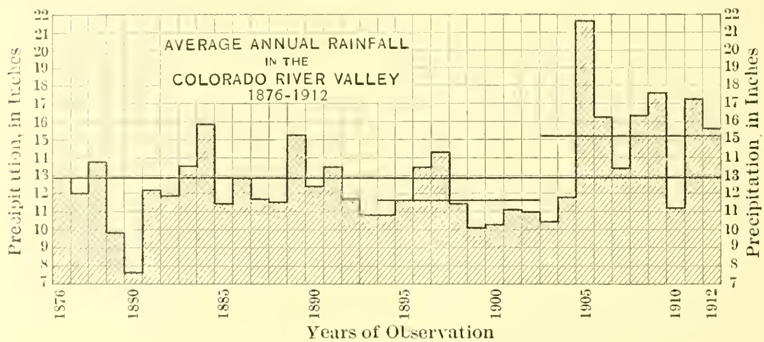


FIG. 37.

servations within the drainage area appeared to be available, therefore Denver, Salt Lake City, and Cheyenne were used. These stations are probably indicative of conditions within the area under consideration. For the central and southern portions, the records of Prescott, Tucson, Yuma, Phœnix, and Flagstaff were taken. The average precipitation for the entire period is 12.92 in., shown by the heavy line. That for the period 1894 to 1902 is 11.73 in., and for 1903 to 1912 is 15.20 in. These latter amounts are indicated by the shorter and lighter lines. Thus it appears that although the run-off in the latter half of the period covered by Table 3 is about 2.4 times that of the first half, the increase in precipitation was only 1.3 times. Furthermore, it appears that the sudden increase in precipitation did not occur until 2 years after the change indicated by the table. There is no doubt that there is considerable difference in the run-off of the two periods, but it is not nearly as great as Table 3 indicates. The explanation is clear: In November, 1902, careful and systematic gaugings of the

stream were inaugurated at Yuma by the Reclamation Service, and the data since collected are dependable. The data prior to that time are based on a rating-curve constructed from so few observations that the results are worthless. The first half of Table 3 should not have been presented. Mr. Sellew.

Available Storage Sites.—Having indicated in a general way the volumes of storage required to attain various degrees of control of the Colorado at Yuma, the next inquiry relates to the existence of such storage. The writer has not the advantage of having made a personal examination of the upper river and its tributaries, hence this portion of the discussion must be limited to the data found in published reports to which he has access. The only authentic data found are contained in the various annual reports of the Reclamation Service, to which reference is made for more complete information. The first reference is to the Second Annual Report, page 123, "Investigations on Colorado River", where mention is made of the various schemes examined. On page 132 data are given of a dam site at Williams River, where 1 300 000 acre-ft. of storage may be created. The statement is made that investigations of a site at Bulls Head were abandoned because of poor foundation. A 100-ft. dam at this point would provide storage for 845 000 acre-ft. The lower end of Pyramid Canyon was examined. Surface conditions appeared to be favorable for a high dam, but no suitable foundation was found. If it were possible to build dams at these points, the scheme would be impractical because the reservoirs would soon be filled by the deposition of sediment, which will appear from data given later. Reference is made to this difficulty in the Second Annual Report, page 155, where we may read:

"In connection with this question of the study of reservoirs which silt up, it seems most necessary that a thorough investigation be made of the reservoir possibilities upon the head-waters of the Colorado, above the points where the streams carrying large quantities of silt enter, so that permanent reservoirs may be established in anticipation of the time when all the irrigable lands along the Colorado will be under ditch."

Thus the Second Annual Report shows no available storage sites for the permanent control of the river.

The Third Annual Report, page 70, has the following:

"Colorado River Storage Projects.—As it has been thought necessary to store water in the Rocky Mountain region for the regulation of the flow of the Colorado River, with a view to the reclamation of about 600 000 acres of land along the lower Colorado in Arizona and California, certain reservoir sites have been withdrawn in western Colorado, two of these being on Grand River, in Grand County, and two on Yampa River, in Routt County. Preliminary examinations have been made of all these sites, and some of them have been surveyed."

Mr. Sellew. On page 217, the Kremmling site is credited with a capacity of 1 500 000 acre-ft. At Grand Lake about 23 000 acres are reported as withdrawn, but no capacity is given. Two sites on the Yampa were located and 100 000 acres withdrawn, but no capacity is given, as the data were not complete.

In the Fourth Annual Report, page 121, it is stated that at the Kremmling site more than 1 000 000 acre-ft. may be stored by a dam 180 ft. high, and that a 230-ft. dam would impound 2 200 000 acre-ft.

On page 124 reference is made to further examinations on Grand River above the Kremmling site, where it is stated that: "Few reservoir and dam sites worthy of investigation were encountered during the reconnaissance."

Reference is made to:

Windy Gap site—capacity	100 000	acre-ft.
Lehman site—capacity	24 000	" "
Grand Lake—capacity	140 000	" "
	264 000	acre-ft.

The first site is crossed by a railroad; would require a large dam; and 75% of the area flooded would be ranches already under cultivation. The second site would cover valuable land under cultivation, and is of slight consequence in size. Regarding Grand Lake, the report says, on page 127:

"Considered as a site for storing irrigation water for use along the lower Grand and Colorado rivers, Grand Lake, as will be readily seen from the foregoing, would be of little value."

It appears, then, that this report shows no storage sites of value except the Kremmling, with a possible capacity of 2 200 000 acre-ft.

The Fifth Annual Report states, on page 114, that the field work in connection with reservoir sites on the Colorado was brought to a close late in 1905, and no additional examinations have been made since those described in the Fourth Annual Report. The Sixth Annual Report gives no additional information regarding this matter. On page 57 of the Seventh Annual Report reference is made to investigations for storage reservoirs along the Green River and its tributaries, from which the following is obtained:

Flaming Gorge site—capacity	350 000	acre-ft.
Island Park site—capacity	150 000	" "
Browns Park site—capacity	2 500 000	" "
	3 000 000	acre-ft.

No additional information is contained in the Eighth, Ninth, or Tenth Annual Reports.

Taking from the foregoing those sites which appear to be available for storage on the tributaries of the Colorado, we have: Mr. Sellew.

		Capacity in acre-feet.
Grand River:	Kremmling site	2 200 000
Green River:	Flaming Gorge site	350 000
	Island Park site	150 000
	Browns Park site	2 500 000
	Total	5 200 000

Railroad interests have secured control of the Kremmling site, therefore its future development may prevent the creation of reservoirs of any magnitude. Such a result would remove the only known reservoir capable of controlling the Grand River, and would reduce the known available storage for uses on the Lower Colorado to 3 000 000 acre-ft., all confined to the Green. This is but little in excess of that required for minimum-year control in the interests of irrigation, and furnishes little relief in the years of maximum flow, when, as previously shown, 7 000 000 acre-ft. must be stored in order to prevent overflow, unless dependence is placed on levees. The foregoing data are not sufficient to determine with accuracy the feasibility of reservoir control on the Colorado River. Before conclusions can be properly drawn, investigation should show the total available storage, its geographical distribution, and whether it is placed so that it will control the run-off from the various parts of the catchment area. Should sufficient storage exist to save the surplus of the maximum years for use in years of low flow, the irrigable area along the Colorado could be increased correspondingly.

The Quantity of Silt.—As previously stated, reservoirs created on the lower portion of the stream would very soon fill with deposits, as the Lower Colorado is a notorious silt carrier. Professor R. H. Forbes* says, regarding Colorado silt:

"Beginning with January, the regular low winter water averages about 62 parts of sediment in 100 000 parts of water during January, February, and March. These sediments are probably chiefly the result of erosion upon the upper river channel, rather than surface sweepings into the drainage. During April, as the river rises with the melting snows of its highest water-shed, and its erosive power increases, the sediment rises to an average of 112 parts; during the highest waters of May and June, to an average of 374 parts; decreasing with the lowering waters of July and August to an average of 122 parts. The summer rain-storms of September, October, and November, partly within Arizona, bring the silt to its maximum (2 072 parts, Oct. 8-13), after which the quantity sinks to the normal amount for winter waters, averaging 151 parts in 100 000 during the two months ending Jan. 24, 1901.

**Bulletin No. 44*, Arizona Experiment Station, pp. 198 and 200.

Mr. Sellew. "* * * The average acre-foot of Colorado river water for the year of observation carried 7 291 pounds of silt, varying from 1 182 to 56 410 pounds as shown by 6-day composite samples.

"On the basis of the profile constructed from available data for the volume of flow of the Colorado, and of the year's silt determinations made in this laboratory, it is estimated, conservatively, that the river during 1900 brought down about 61 000 000 tons of sedimentary material, which, condensed to the form of solid rock, is enough to cover 26.4 square miles one foot deep; or, to make 53 square miles of dry alluvial soil one foot deep; or, to make about 164 square miles of recently settled, submerged mud one foot deep, reckoning the whole amount of mud for the year to average 6.2 times the bulk of the solid sediment."

From these extracts it appears that in 1900 the volume of sediment varied from 0.062% in January, February, and March, to 0.374% in May and June, and that following desert rains it was as high as 2.07 per cent. An area of 164 sq. miles of mud 1 ft. deep equals 105 000 acre-ft. Since May, 1909, the Reclamation Service has observed the quantity of sediment passing Yuma, the results being as shown in Table 30.

TABLE 30.—QUANTITY OF SEDIMENT IN COLORADO RIVER.

Year.	Total discharge, in acre-feet.	Number of silt observations	Average percentage of silt, by weight.	Acre-feet of submerged mud annually.
1909......	25 967 577	8	0.65	321 010
1910......	14 332 692	22	0.50	136 660
1911......	17 839 245	52	1.15	391 115
1912......	18 357 686	64	0.756	265 734
Total..	....	....	....	1 114 519

The observations of 1909, although few in number, covered the period from May 26th to October 7th, and should give a fair average. Those of 1910 extended from April 19th to December 31st. In 1911 they extended from January 11th to December 28th, and in 1912 from January 4th to November 30th, the discharge being estimated to December 31st. As shown in Table 30, the total quantity of sediment passing Yuma during the last 4 years, when measured as submerged mud, has exceeded 1 000 000 acre-ft. This proves conclusively that reservoirs for the permanent control of this stream must be placed on the head-waters where clear water exists.

The author shows in Table 2 an approximate storage of about 10 000 000 acre-ft., and although he qualifies the table by stating that

it is "theoretically possible" and would have to be reduced for "commercially feasible developments," the table is so at variance with facts as to be misleading in its present form, and likely to lead uninformed persons to erroneous conclusions. Storage on the Gila is out of the question, as this stream carries more silt during its flowing season than the Colorado. Little Colorado, Bill Williams Fork, and San Juan must be eliminated for the same reason. The Kremmling site is in the hands of interests foreign to irrigation or storage development. The only site of value contained in Table 2 is Browns Park on the Green, which has a capacity of about 2 200 000 acre-ft., as previously indicated. This analysis reduces the author's "theoretically possible" 10 150 000 acre-ft. to 2 200 000; reservoirs which will silt up in a few years or are controlled so that they cannot be developed, are not in the "possible" class. Mr. Sellew.

There are some statements which the writer cannot reconcile with the facts as they have come under his notice. A water supply of 1 000 000 acre-ft. is said to be ample for 1 000 000 acres in the San Juan country, though in reality two or three times that quantity will probably be necessary. The area of the Yuma Project is some 130 000 acres, instead of 90 000. Table 4, showing mean monthly discharge from 1894 to 1911, is valueless, for the results from 1894 to 1903 are based on erroneous data, as previously shown.

RISE OF BED.

The data in regard to the rise of bed at Yuma are based on observations made largely by the Reclamation Service, but are not considered by the writer as sufficiently definite to justify the conclusions. Although the advance of the delta into the Gulf will tend to raise gauge heights at points above, certainly many more observations are necessary before definite relations can be established between the rise on the Yuma gauge and the memory of a steamboat captain.

There is at least one other cause for variation in gauge height at different points on the stream, which is entirely dissociated from any advance into the Gulf. There are two points on the lower river in this vicinity which may be regarded as definitely fixed. They are Laguna Dam and the conglomerate hills at Yuma between which the river passes. The floods of 1909 and 1912 carried substantially the same quantities at their crests, 150 000 sec-ft., but the difference in the elevation of water surfaces between the Yuma gauge and Laguna Dam was 4 ft. greater in 1912 than in 1909. The casual observer might think that this indicated a rise in the bed of the stream. It is found, however, that the slope per mile was substantially the same, being 1 ft. in 1909 and 1.04 ft. in 1912. The rise is not due to any silting up of the channel, but is plainly caused by the meanders of the river.

Mr. Sellew. The records show that the distance from Laguna Dam to Yuma has varied as follows:

In 1856	13	miles.
1895	$11\frac{3}{4}$	"
1902	14	"
1909	$13\frac{3}{4}$	"
1911	$15\frac{1}{4}$	"
1912	17	"

Some of these various meanders are indicated on Plate L. The river in 1912 between the points mentioned was $3\frac{1}{4}$ miles longer than in 1909, and this accounts for practically all the increased difference of elevation in the water at the two points. The same phenomenon is true on the lower river, and, assuming a given elevation at the Mexican boundary, the increase in length of the stream due to meanders will give an increased height on the Yuma gauge, whether the river is advancing into the Gulf or not.

The author's description of the works of the Yuma Project and remarks relative thereto, indicate unfamiliarity with the subject. A very excellent account of the work at Laguna Dam may be obtained from articles* by the Resident Engineer, Edwin D. Vincent, M. Am. Soc. C. E. The Colorado Siphon has been described by the writer.† These cover the main features of the work. Levees and canals are so common, and those at Yuma present no unusual conditions, that description appears to be superfluous.

The author's statement, that a rock fill such as was used for closing the crevasses which he discusses could have been used in place of Laguna Dam, is open to serious doubt. What he built has been used as a levee, which purpose it answers admirably. How it would act as an overflow weir, carrying 150 000 sec-ft., even if covered with concrete, is problematical. In addition to a concrete top, crest and foot-walls appear to be necessary to prevent the top from being undermined, and, if these are provided, the resulting structure will not differ materially from Laguna Dam either in design or cost.

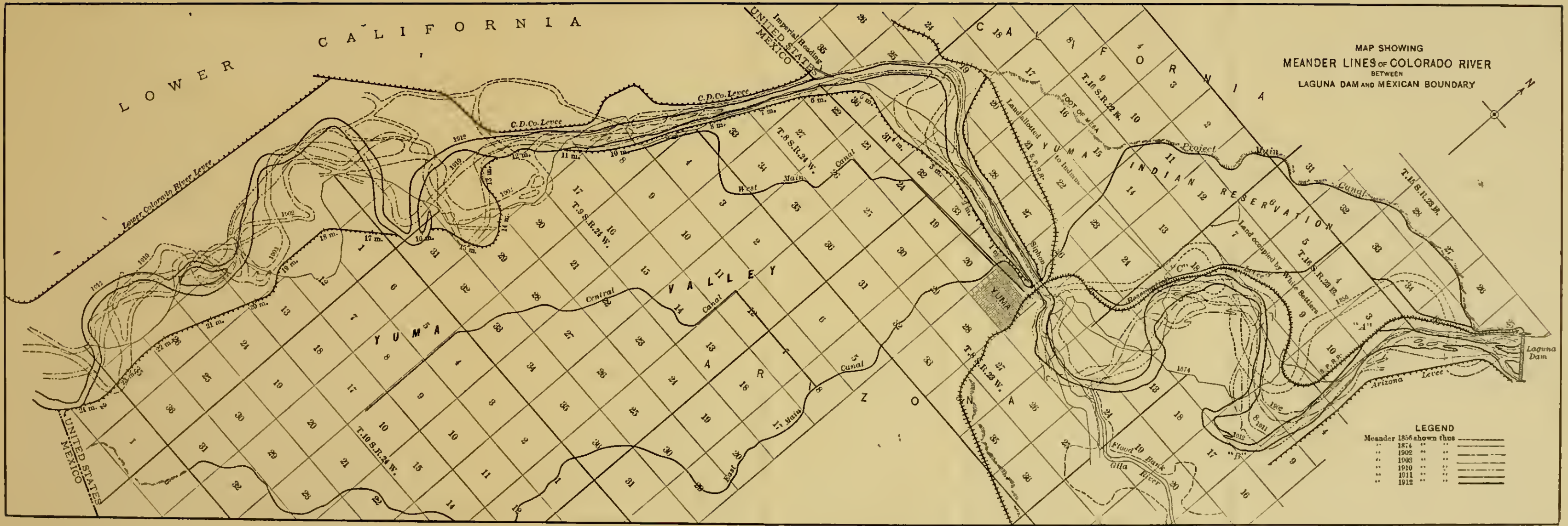
Levees.

The statement that the flood of December 7th, 1906, caused levee failures on the Yuma Project due to the absence of muck-ditches, is not correct. In October, 1906, the writer took charge of the Yuma Project, and his first act was to order muck-ditches under all new levees. The slight damage done here by the flood of December 7th, was caused by burrowing animals and the overtopping of work which was incomplete.

* *Engineering News*, February 27th, 1908, and June 10th, 1909.

† *Engineering News*, August 29th, 1912.

PLATE L.
PAPERS, AM. SOC. C. E.
MARCH, 1913.
SELLEW ON
IRRIGATION AND RIVER CONTROL,
COLORADO RIVER DELTA.



Mr. Sellew.

Regarding the advantages and disadvantages of land-side and river-side borrow-pits, the writer has fully expressed himself,* so that it is not necessary to comment further. Particularly is this so when the author states that he "realized that this was not in accordance with the usual practice." Based on a personal examination made a few hours after the break of December 5th, 1906, the writer hazards the opinion that the damage caused by a head of 15 in. against the levee was due primarily to the presence of land-side borrow-pits, although small quantities of water came under the levee itself in many places due to the absence of a muck-ditch and the improper grubbing of the foundation.

Regarding the loan of a grading camp by the Reclamation Service, which outfit the author declares did work at excessive cost, the following is submitted:

Immediately following the break, in December, 1906, the writer discussed with Mr. Cory the feasibility of assembling a sufficient force to construct levees required before the coming of the annual freshet. At that time it was not known how much levee work would be done, as the surveys were incomplete. Consequently, no contracts were let, and the size of the outfit which could be immediately available was not known. Because of various interests at stake, the writer suggested that, to assist in overcoming the difficulty, one camp of Government stock then operating on the Arizona levees be transferred for the use of the California Development Company. Mr. Cory welcomed this suggestion; authority was secured from the Department; and the camp was immediately transferred. It was supposed when the transfer was made that the camp was to work for the California Development Company and not for a contractor. However, shortly after the move was made, the superintendent of the camp informed the writer that he was working under the direction of a contractor, and it was next to impossible for him to obtain the proper supplies to keep his camp up to efficiency. A visit to the camp showed the facts to be substantially as stated, and a personal appeal was made to Mr. Cory to remedy the matter. This was remedied, in a measure, but conditions continued to be such that, as soon as it was evident that the camp was not necessary to secure the completion of the levees in advance of the annual flood, the men and equipment were brought back to the United States.

The camp that was loaned the C. D. Co. had been at work for considerable time prior, for the Reclamation Service, and continued to be thus employed for several years thereafter. It was one of the most efficient camps which the Service has had on levee construction. There have been completed on the Yuma Project about 50 miles of levee, aggregating more than 2 600 000 cu. yd. of excavation, the ac-

* *Engineering News*, February 15th and April 11th, 1912.

Mr. Sellew. complishment of which compares most favorably in every way with any levee work which has come under the writer's notice. Criticism of Government work seems to be a general pastime of people who are not responsible for it, and certainly if a contractor desired to place a Government camp in an unfavorable light, Mr. Cory granted him a most excellent opportunity to do so.

Under the head of "Criticism of Levee Work Done" the author shows that after a Consulting Board from the Reclamation Service had advised on the methods of levee construction, their recommendations were ignored and a second Board was necessary before the recommendation as to the location of borrow-pits was complied with. The author states that the use of the checker-board system of borrow-pits increased the cost of the work about $6\frac{3}{4}$ cents per cu. yd. Experience on the Yuma Project has shown that an increase of 2 cents per cu. yd. is ample to cover this kind of borrow-pit, and the writer believes that the protection thus afforded fully justifies the additional expense.

International Negotiations.

International agreements are necessary to a comprehensive plan for the control of the Lower Colorado. Such agreements must be preceded by diplomatic action, and while such negotiations may be in progress, discussion of this phase of the matter appears to be inopportune.

The Abejas Diversion.

The author is correct in the statement that this diversion has had little effect on river levels at the entrance of the Imperial Canal. The observations show that it has had no effect whatever. The writer has little first-hand knowledge of the work at the Abejas, carried out under the direction of John A. Ockerson, Past-President, Am. Soc. C. E., so does not feel qualified to discuss it. He wishes to state, however, that the delay in starting the work, due to causes over which Mr. Ockerson had no control, which exposed the operations to the menace of the flashy winter floods, had much to do with the progress. It is believed that the Ockerson plan to levee the river to the range of tidal influence is the correct one; until this is done and the levees are made secure against the attack of the current, the Lower Colorado cannot be regarded as under control.

The Real Problem.

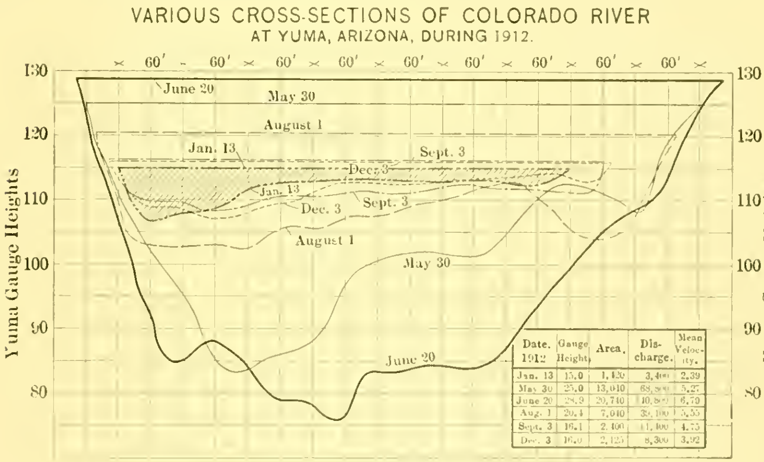
The author concludes:

"Because of the various successful and unsuccessful work done in the region, the engineering features of irrigation and river control along the Lower Colorado are now understood, and engineering construction methods are thoroughly developed.

* * * * * *

"The Colorado River Delta now presents no unusual unsolved engineering difficulties; its problems are chiefly matters of statecraft, in both river control and irrigation." Mr. Sellew.

After more than 6 years of service, during which time about 45 miles of the lower river has been in his charge, the writer wishes to state that, in his judgment, the real problem of this river is not generally understood. This problem, for the solution of which the author offers no aid, is bank protection. Construction methods for its accomplishment are not developed, and it cannot be attained by statecraft, however high the talent engaged. Relief from this menace will come only from hard work and a liberal expenditure of money on the immediate banks. When levees are undermined while the stream is below bank-full stage, it is clear that many years must elapse before a river of this magnitude will be controlled by reservoirs in such a way that caving banks will be eliminated. A plan by which these banks may be made stable will now be outlined but, before proceeding with it, a short sketch of the flood of 1912 may be of interest.



Date. 1912	Gauge Height.	Area.	Discharge.	Mean Velocity.
Jan. 13	15.0	1,420	3,400	2.39
May 30	25.0	13,040	68,900	5.27
June 20	28.9	20,740	140,800	6.79
Aug. 1	20.4	7,040	39,100	5.55
Sept. 3	16.1	2,400	11,400	4.75
Dec. 3	16.0	2,125	8,300	3.92

FIG. 38.

LOWER COLORADO FLOOD OF 1912.

The annual freshet of 1912 first made its appearance on the Yuma gauge on March 12th, the reading then being 117.8 ft. and the discharge 12 800 sec-ft. It continued to rise until June 22d, when the gauge was 129.1 and the discharge was nearly 150 000 sec-ft. During this rise the erosive action of the current was causing radical changes in the bed and banks of the stream, as may be seen by Fig. 38, showing conditions at the gauging station. It is there seen that at low water the width of the river was 420 ft., its average depth about 3½ ft.,

Mr. Sellew. maximum depth 8 ft., and the area of the flowing prism 1 420 sq. ft. At the crest these dimensions had become, width 600 ft., average depth 35 ft., maximum depth 53 ft., area 20 740 sq. ft., the discharge having increased 43 times and the area about 15 times. This increase in area, of 19 320 sq. ft., is made up of 7 440 sq. ft. caused by the rise on the gauge and 11 880 sq. ft. (60%) which was the increase in section due to scour. These modifications in cross-section were probably maximum at the gauging station, where the width of the stream is restricted; but similar changes, although less in magnitude, are characteristic of the lower river.

During the rise, extensive meanderings were in progress at various points, the first attack which required defense being between 10½ and 12 miles below Yuma, on the Arizona side, where the levee had been cut out by the river in 1909. During 1909 and 1910 efforts were made to hold this portion of the bank with fascines and other light brush protections, but such work was soon carried out, and resort was then had to permeable dikes of piles and brush. About seventeen permeable dikes have been built, beginning in May, 1911. Some of these dikes are shown on Figs. 1 and 2, Plate LI. At this point the Steamer *Searchlight*, with a pile-driver barge worked throughout the flood of 1912 building dikes and defending the bank between them with fascines. A temporary levee was also thrown up when the river threatened overflow. The efforts here were successful, the dikes holding well and preventing further recession of the bank line, and the temporary levee removed all danger from overflow.

The first attack on the Reservation levee was at the point, *C*, on Plate L. At the beginning of the rise, the bank at this point was some 800 ft. from the levee, but on June 15th bank caving set in, and on June 17th, when the river was within 150 ft., defensive operations were commenced. As all our floating plant was engaged below Yuma, the fight here had to be made from the shore. When the stream went overbank the bottoms were flooded to within 150 ft. of the levee, making defensive operations impractical until the caving reached this point. Fascine mats weighted with sand sacks were hung over the face of the bank, but, as the water was more than 30 ft. deep, such defenses would not reach the bottom, and, although our work delayed the advance of the stream, it was slowly undermined, demonstrating that a strong revetment would be necessary to hold the river in check. Conditions along the front at this point are shown on Fig. 3, Plate LI. Fortunately, the Reservation levee furnishes the location for the Potholes Branch of the Southern Pacific Railroad, therefore, rock, which was the only material that would save the day, could be delivered at the site of the works and dumped directly into place. Rocks weighing from 1 to 6 tons were ordered from the Declez grante quarries some 200 miles west of Yuma on the main line of the Southern Pacific. The railroad ex-

Fig. 1.—Permeable Dikes.

Fig. 2.—Permeable Dikes.

Fig. 3.—Conditions Along the Front.

pedited the shipment, and within 36 hours after the order was placed the first train load, consisting of fifteen cars, was being thrown into the river. At this time the river had cut to the toe of the levee slope, the water there being 32 ft. in depth, therefore the rocks as rolled from the cars could be directed to the point where they would accomplish the most good. Considerable brush and thousands of sand sacks were mixed with the rock, also a limited quantity of spoil which was available from the quarries at Laguna Dam. As the flood came up to crest, the attack extended for a length of about 900 ft. Several bad sand boils appeared at the inner toe. Fig. 1, Plate LII, shows one of these boils "hooped" with sand sacks. This menace became so serious in some instances that at one time the destruction of the levee appeared to be only a matter of a few hours, and arrangements were made to confine the overflow in case the levee went out. About 500 ft. to the rear of and parallel to the levee is one of the canals of the Reservation distribution system. The outer bank of this channel was hastily connected with the levee above and below the threatened portion by sandbag dams (Figs. 2 and 3, Plate LII), the canal bank between these dams being raised to about 1 ft. above the river level. Sand bags were necessary as the ground, because of the seepage, was under several inches of water, and stock could not be worked upon it. These dams and the canal bank would have confined the overflow to this area had the levee gone out. It was also intended to flood this enclosed area to a depth of 2 ft. or more to check the sand boils, water for this purpose being supplied from the canals, but the boils were checked by a liberal use of sacks along the front before such action was necessary. Figs. 1 and 2, Plate LIII, show the character of the rock used for protection, and Fig. 3, Plate LIII, is a view of the Steamer *Searchlight* engaged in dragging the larger rock from the cars with her steam capstan, the boat having been temporarily transferred for this purpose from the work below. Mr. Sellew.

The flood crested at about 1 A. M. on June 23d, and then fell rapidly. By June 25th there was no further danger of overflow, but dumping the spoil, in order to blanket the outer slope and reinforce the levee, continued until August 18th.

While this attack was in progress, another was initiated by the river at the point, *A*, on Plate L. Here the caving banks came to the levee toe with a water depth of 30 ft., and the defense was practically a repetition of the one just described.

On June 25th, the river having fallen below the overflow line, shipments of Declez rock were stopped; a steam shovel was placed in commission at Laguna Dam, and a work train and crew were obtained from the Southern Pacific Company, which, with assistance from our dinky engines, continued to dump rock and spoil at the points, *A* and *C*, until August 18th, when 18 000 cu. yd., including 1 300 cu. yd. of

Mr. Sellew. Declez rock, had been placed. On this date the threatened levees were safe from further attack, and the work was stopped. At the present time the river lies against the rock slope of the levee at *A* for 1 500 ft. and at *C* for 1 000 ft. Should these conditions remain until the next flood, further rock revetment will probably be necessary immediately above and below each of these points, therefore steam shovels, cars, and engines are being placed in condition to do this work if required.

While the attack on the Reservation levees was going on, rapid bank caving was also in progress on the Arizona side below Yuma. The work at the 12-mile post has already been briefly described. At the 16-mile post the river was 600 ft. from the levee on May 25th. On June 1st it had moved out to 1 300 ft. On June 10th it had returned to within 500 ft., on June 20th about 300 ft., and on June 25th it cut through the levee. When the caving had approached within 300 ft. of the levee, provisions were made to protect the work. There was no floating plant available for this protection, and, had there been, no effective use of it would have been possible, as the depth of water and velocity of the current made the construction of spur dikes impracticable. As there was no railroad on the levee, rock could not be obtained, as the roads to the nearest quarry, 12 miles away, were impassable for heavy loads. The only defense that could be offered was to protect the bank by fascine mats, there being plenty of suitable willow brush near at hand. The force employed, consisted mainly of Cocopah Indians, was rather indifferent labor, but by diligent efforts the rate of caving was delayed so much that, when it reached the levee on June 25th, the river had fallen below the natural bank level and no overflow occurred.

Shortly after the cutting out of the levee the river swung away for several hundred feet, and later came back for a portion of this distance, the present position of the river being shown by the meander line on Plate L.

About July 1st, the river being well within its banks, the patrol which had been maintained during the flood was removed. On this date no serious cutting appeared at any point between Yuma and the International Boundary Line, on the American side of the stream.

On August 10th it was found that the river had cut through the levee at the 24-mile point, but the bank here being above the river level, no overflow had occurred. On June 20th, or a few days prior to the crest of the flood, the bank at this point was 1 300 ft. away from the river, and there appeared to have been no material change on July 1st, when the patrol was taken off; thus it appears that the caving here must have been extremely rapid.

All work of bank protection was continuous, day and night, while the danger of overflow existed. During this period the temperature in the day averaged 106° and at night about 73 degrees. Because of the

FIG. 1.—SAND "BOILS" HOOPED WITH SAND SACKS.

FIG. 2.—SAND-BAG DAM.

FIG. 3.—SAND-BAG DAM.

extreme heat, the work was very hard on the men, and at times our efforts were greatly hampered by lack of labor. All the Indians in the vicinity were pressed into service. Wages were temporarily increased to attract idle men in the Town of Yuma, and on the Reservation the settlers, whose lands were threatened, turned out practically to a man and fought valiantly in the defense of their homes. Mr. Sellew.

FUTURE BANK PROTECTION.

The work of bank protection should be directed:

(1) Toward holding existing levees and regaining lost ground, in order that work which has been destroyed may be rebuilt near its original position, thus maintaining as nearly as possible the irrigable area;

(2) To cause the river to occupy a position about midway between the levees in order to prevent, if possible, the erosion of one season from reaching the embankments; or the river slopes of the levee should be made impregnable to river attack.

It is also desirable, in order to prevent meandering—which, as shown previously, may result in a rise in plane—to confine the stream to the shortest route consistent with a fall of about 1 ft. per mile.

Regarding the recovery of lost ground, the writer is aware that the building of levees on their old locations has not been generally practiced. The following, relating to the Mississippi, is from *Occasional Papers No. 41*, Engineer School, U. S. A., page 278:

"*Location.*—The two objects of levees, protection of land and concentration of flood discharge, will be most efficiently obtained by building the levees as close as possible to the river bank, and, as owing to the general slope of the bottom land, the banks are usually higher than the land farther back, their location close to the river satisfies generally the two conditions of maximum usefulness and minimum cost, and if the river were straight and its banks stable no better location could be sought; but the river is not straight and the banks cave rapidly, and a location close to the river therefore has some disadvantages. On some of the long points of the river a levee following the bank will have, compared with a line across the point, a length not justified by the small additional amount of land protected. In a rapidly caving bend a levee close to the river bank must in a short time be destroyed. Complications are also frequently introduced by the existence of the old river lakes caused by cut-offs. The short and direct line would pass between the lake and the river, but such a location requires usually a high and large levee and involves danger from the nature of the soil recently deposited in the old river bed and the probable existence of permeable sub-strata liable to be washed out by hydraulic pressure during floods. On the other hand, the line around the lake, though safe, is apt to be of extreme length.

"The question of levee location is, therefore, not a simple matter and should receive careful consideration, but such consideration has, un-

Mr. Sellew. fortunately, not been always given to it. Most of the existing levees have grown up gradually by the incorporation in the main system and successive enlargement of old levees built by private parties for their own protection, and the location of these old levees, having been made without any reference to a general system, is, of course, in many cases, faulty. Even where the levee has been built entirely anew the location has been most usually fixed by the local boards, who are, of course, subject to local influences, and many levees have thus been located too close to a caving bank, in order to include special farms.

"But even when a new levee is to be located with reference only to its general use, the question is not an easy one. If the banks of the river cave with any regularity, the cost of a new levee and its life in any location could be balanced against the value of the additional protection obtained by this location. But the caving does not go on with any sort of regularity; some places that may not have caved for years will suddenly begin to cave rapidly, and at other places where caving has long been going on the caving will sometimes somewhat suddenly cease. In placing levees near caving banks, therefore, a careful study must be made of the locality and of the river for some miles above, and, in general, it is considered a good location for the levee if this study indicates a probable life of about twenty years."

Although 20 years was formerly considered economic practice on the Mississippi, recent developments have modified it, as shown by the following:*

"It is the judgment of the Commission that these revetment works and some others which will be imperative within a short time are required for the preservation of the levee system. Without them a permanently completed system will be impossible. The increasing height and cost of the levees and the rapid development of the country protected by them greatly augment the disaster of crevasses. A break in an embankment 5 or 6 feet high resulting in the thin overflow of a sparsely populated country is a small calamity compared with a break in a levee 20 feet high, discharging over a thickly populated country threaded with railroads and filled with towns and factories. Owing to the increased height now necessary to control the floods the expense of replacing an undermined levee by a new line going far around a lake or impassable swamp is enormous. The point of development has come at which the only economical way to maintain the levees is to hold the banks against caving—not everywhere, but in all places where the special conditions exist which have just been described. This is a distinct stage of the work only recently reached."

The area protected by the Colorado levees will be occupied by farms of rather small area, and consequently a comparatively large population will be dependent on them. Intensive farming will be practiced, and the preparation of the land will be expensive, therefore it appears that overflow should be prevented by all possible safeguards. The integrity of the levee system will depend principally on bank protec-

* Report of Mississippi River Commission, Appendix PPP—Chief of Engineers. 1908, p. 2655.

FIG. 1.—SIZE OF ROCK USED FOR PROTECTION.

FIG. 2.—LEVEE PROTECTED BY ROCK.

FIG. 3.—STEAMER "SEARCHLIGHT" DRAGGING LARGE ROCKS FROM CARS.

tion. There are two general methods by which this may be accomplished: (1) to give the bank a revetment which will protect it against erosion; (2) to keep the main current away from the banks, so that erosion cannot occur. Mr. Sellew.

Revetment may be continuous throughout the river, or salient points which are strongly attacked may be treated first, the work being extended as occasion demands. The most extensive revetment in the United States, if not in the world, is to be found along the Mississippi River and its tributaries, where the General and State Governments and local levee boards have engaged in protective works for many years. The standard revetment consists of a woven brush mat or fascine mattress protecting the bed of the stream for a short distance from the shore and the side of the channel below low water. Above low water, where brush deteriorates rapidly, the bank is graded to a uniform slope and paved with rock. Such work is very expensive.

The magnitude of bank protection and river control on a large stream is seen in a "Report by a Special Board of Engineers on a Survey of the Mississippi from St. Louis to its Mouth,"* with a view to obtaining a channel 14 ft. deep and of suitable width. On page 52 it is stated that up to the date of the report, 1909, there had been expended by the General Government on the Mississippi from the mouth of the Missouri to the head of the Passes more than $88 000 000. In addition, there had been some millions spent by the riparian States and local levee boards.

Reference is again made to *Occasional Papers No. 41,* Engineer School, U. S. A., being "A Resumé of Operations in the First and Second Districts of the Mississippi Improvement, 1882-1901." This material, which covers the work done by the National Government under the direction of the Army Engineers, has been compiled in detail by E. Eveleth Winslow, Major, Corps of Engineers:

(From page 45): "It is now considered settled that in a plan for the permanent improvement of the channel by the contraction of its low-water width, the work of bank protection should precede the permeable dikes and other structures of that class, and that the latter may be regarded as supplementary in functions and as necessary only in a minority of cases."

(From page 47): "It was recognized from the beginning of the Commission's work that the construction of works of bank protection and channel contraction in detached reaches would be subject to influences more or less unfavorable from changes in unprotected bends above. In commencing work in the detached reaches of Plum Point and Lake Province, the Commission was not oblivious of this fact, but was influenced by other considerations which seemed to overbalance this disadvantage, and it has not since seen good reason to doubt the wisdom of that decision. But the experience gained in those reaches

* Doc. 50, 61st Cong., 1st Sess.

Mr. Sellew. emphasized the fact that it is highly important in any project for general and permanent bank protection to leave no unprotected reaches above the work done from changes in which new conditions may be produced in improved parts of the river. And it is considered by the Commission that in undertaking a complete and permanent system of bank protection the work ought to begin at the upper end of the river at or near Cairo and be carried thence down-stream with substantial continuity. * * *

"It is considered that the annual cost of maintenance of completed work could not be estimated at less than ten per cent of the original expenditure."

(From page 233): "The object of bank revetment is to protect the banks of the river from destruction by the currents. The active agency in this bank destruction is erosion, and bank revetment must therefore prevent this erosion. Perhaps the easiest and most obvious method of doing this is by laying over the exposed surface of the erodable bank a non-erodable covering, and this is the method adopted. This non-erodable covering must, of course, be fairly durable, must have sufficient strength to withstand any strain put upon it, and must be free from interstices through which scour might take place.

"Owing to the fluctuations in the river, the different parts of the bank are subjected to different conditions. Below the low-water line the bank is always wet; above that line sometimes wet and sometimes dry, and this difference in conditions allows, if it does not require, that the portion of the bank above and below low water be treated differently.

"Above low water the bank can be seen and the non-erodable covering can be laid with ease, and it has been found that here a properly laid stone pavement forms an efficient protection. The individual stones of this paving must, of course, be so large that they cannot be moved by the current, and it was found by experience that stones of this size are not of themselves sufficient, as through their interstices scour could take place when the bank was sandy and the current strong. This has been prevented by first covering the bank with a layer of spalls or crushed rock of such thickness and closeness as to prevent scour through them and to hold this down with a layer of larger stone. Such protection, when carefully laid, has been proven to be all that is required for the upper bank.

"Below low water the conditions are different. As it is impossible to construct the non-erodable covering actually in place, the next best thing is done, constructing it on the surface of the water above where it is to lie, and when constructed sinking it. In this way the continuity of the covering can be assured, and it can be placed exactly where it is needed. As has been stated, this covering must not contain interstices through which scour can take place, for in such a case not only would the purpose of the covering not be fulfilled, but by such scour its own eventual destruction would be assured.

"The covering actually used has been a mattress of brush and the only kind that has been found to prevent scour in very rapid currents is the fascine mattress. In moderate currents another type of

mattress might do, but as changes in current conditions are always likely to happen the fascine mattress only should be used." Mr. Sellew.

(From page 256): "In the early work brush was used as a covering for the upper bank, but, subject as it was here to alternate submergence in the water and exposure to the atmosphere, it decayed rapidly and possessed little strength after about two seasons, and it was this rapid decay of the brush permitting it to be easily broken up and exposing the bank to erosion, that led to its disuse and the substitution for it of an all-stone covering."

(From page 260): "It is probably safe to estimate that a fascine mat built of small brush will be effective for at least twenty-five years and that when a good proportion of large brush is used its life will be considerably extended."

Applying these principles at Yuma, revetment should begin at Laguna Dam and proceed continuously down stream, leaving no unprotected caving banks above completed work.

The portion of the revetment lying above low water should be constructed by grading the bank to about a 3 to 1 slope and paving it with rock. This is necessary, as any brush work laid on this portion of the bank, being subject to alternate periods of submergence and exposure to the air, will deteriorate in about two seasons and be of little use in holding the bank. Below the low-water mark, the sides of the channel and the bottom for 50 ft. from shore should be protected by a fascine mattress. Assuming that the gauge oscillation along the stream is about 12 ft., a slope of 3 to 1 for this height would give an exposed bank of about 38 ft., and, with a stone paving 2 ft. thick, would require 76 cu. ft., or, say, 3 cu. yd. of rock per lin. ft. of bank. By using the quarries at Laguna Dam and Pilot Knob, delivering the stone into barges, and then towing the material into place for use, a fair estimate for the material in place on the bank will be about $2 per cu. yd., or $6 per lin. ft. of bank. In addition, assume that brush mattress will cost about $4 per lin. ft., making a total for revetment of $10 per lin. ft. of bank, or $50 000 per mile. The distance from Laguna to the Mexican boundary is about 45 miles, and probably at least two-thirds of this distance, or 30 miles, will require continuous revetment. At a cost of $50 000 per mile of bank or $100 000 per mile of river, 30 miles of revetment would cost about $3 000 000.

The foregoing quotations from the Mississippi reports state that the annual maintenance of bank revetment should not be taken at less than 10% of the original cost, which means that expenditures for this purpose must be duplicated each 10 years.

S. Waters Fox, M. Am. Soc. C. E., an assistant engineer in charge of some of the Missouri River improvements, states* that 45 miles of the alluvial portion of that stream had been improved at a cost of

* *Transactions*, Am. Soc. C. E., Vol. LIV., p. 280.

Mr. Sellew. $2 500 000, or about $56 000 per mile. The division of this cost was stated by him as follows:

Actual construction	67 per cent.
Care of, repairs, and moving plant	22 " "
Administration	9 " "
Other items	2 " "
	100 per cent.

It is thus seen that, in this particular case, the cost of maintenance was more than double that assumed for the Colorado. Another thing to be considered in the matter of bank protection is the source of the material to be used. The supply of rock in the quarries at Laguna Dam and at Pilot Knob is abundant, but the same cannot be said regarding the brush, which for the best results, should be young, live willow. In *Occasional Papers No. 41,* Engineer School, U. S. A., previously referred to, the following appears on page 46, in an extract from reports of the Mississippi River Commission:

"Another fact now definitely settled is that in the construction of mattresses of the type now employed on a large scale, the supply of material becomes an element of the problem presenting serious difficulty. The area of willow-producing bar necessary to supply material for a mile of fascine mattress 300 feet wide is practically five times as great as that required for an equal length of mattress built in the early years of the work. The quantity of small willow growth used in this form of mattress is so great that the estimate of the yearly supply available along the whole length of the river from Cairo to Vicksburg is not more than sufficient for the making of 15 miles per annum of bank revetment."

The writer is not aware how the growth of willow along the Colorado compares with that along the Mississippi between Cairo and Vicksburg, but will assume for the present that the quantities are about equal. The assumption will also be made that the quantity required for the Colorado will not be more than one-third as much as that on the Mississippi; therefore, if 600 miles of river provide for 15 miles of revetment on the Mississippi, it would provide for 45 miles on the Colorado. If 600 miles of river will furnish material for 45 miles of revetment, then 45 miles of river between Laguna Dam and the International Boundary would provide for about 3½ miles of such work annually. In the 30 miles of stream to which it is suggested the revetment be applied there would be 60 miles of revetted bank, and at 3½ miles per year it would require 17 years to build this revetment with the ordinary growth. This makes no allowance for the annual repairs, which are estimated at 10 per cent. It would also be impracticable, even with the material available, to construct this revetment immediately, not only on account of the constructive difficulties of doing so much work in a short space of time, for the control of the

stream by revetting its banks and contracting its channel must necessarily be slow because the changes which these works bring about cannot occur immediately. On this point the writer quotes from *Occasional Papers* previously mentioned, page 41: Mr. Sellew.

(From the Mississippi River Commission's report for 1889): "This work is still incomplete, although its feasibility and value have, it is thought, been fairly demonstrated. It is, however, well understood now that it is likely to prove a slower process than was at first supposed, for the work cannot be hurried. It must be gradual and progressive."

Seventeen years is entirely too long a period over which to stretch this work, as the protection of the Government's investment and the welfare of the settlers demand that the control of the river be almost immediate.

The radical difference between the high-water and the low-water channels of the Colorado furnish another obstacle to the foregoing form of bank protection. An examination of the diagram (Fig. 38) shows, that while protection of the upper bank by the rip-rap could be readily accomplished, it would be impossible to place the fascine mattress in the proper position. At low water, when conditions are favorable for mattress work, the channel is so silted up that a mattress sunk in position would not even approximate the place it should occupy to protect the flood channel from erosion. At high water, when the channel is scoured out and the bank to be protected is exposed, the depth of water and the velocity of the current are too great to accomplish work of this class successfully.

From the foregoing it appears that mattress revetment below low water, combined with rock revetment above, should not be applied along the Lower Colorado, for the following reasons:

(1) The high first cost and excessive cost of maintenance (10%);

(2) The time required (17 years) for the natural growth to produce the willows necessary for this form of protection;

(3) The impossibility of placing the mattress in its proper position, due to the silted condition of the stream at those seasons when depth and velocity are favorable for such work; and

(4) That in great floods a concentrated attack might destroy a portion of the mattress at a time when its repair or the protection of the bank thus exposed would be impossible.

Rock Revetment.—Another method of protecting the bank would be to make the revetment entirely of rock. At low water the exposed slope would be graded, properly rip-rapped, and at the foot of this slope a large quantity of loose rock would be placed (Fig. 39). As the high-water channel was created by the scour of the current, this loose rock would be undermined and gradually slide into position, making a revetment along the bank thus exposed which would be practically a permanent protection. In approximating the cost of such work, let it

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be assumed that the average depth along the levee during high water will be about 20 ft. It is known that at exposed points where severe cutting has been in progress it has been as much as 32 ft., but, for a general average depth to which revetment should extend, 20 ft. appears to be safe. Suppose that the rock will stand at a slope of 2 to 1; then the length of the revetted slope in contact with the water will be 45 ft., and, if the revetment is 2 ft. thick, the area of the section will be 90 ft. Assuming that, owing to wasteful methods by which this is applied, one-half is lost, then the sectional area of rock per linear foot which has to be placed along the levee is 135 sq. ft., or 5 cu. yd., or 25 000 cu. yd. per mile of bank, equal to 50 000 cu. yd. per mile of river. For 30 miles this would mean the placing of about 1 500 000 cu. yd. of material.

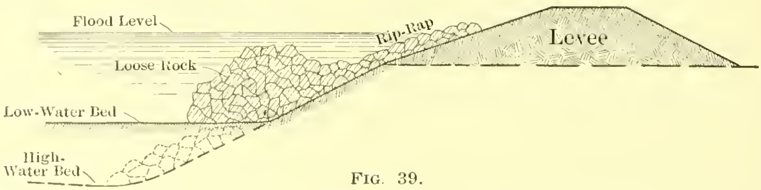

FIG. 39.

Should such a course be adopted, quarries would be opened on each side of the river at Laguna Dam, which, by connecting railroads, would supply the levees within economic reach. Another quarry at Indian Hill, opposite Yuma, would serve for the portions above and below for proper distances, while quarries at Pilot Knob would control the remainder on the California side. From this latter point, a temporary trestle, which appears to be the best arrangement, would connect with the Yuma Valley levee, on the Arizona side of the Colorado. Later, if it was decided to extend similar treatment to the Gila levees, the Arizona quarry at Laguna Dam would control the levee on the north side of the stream, while the south levee could be reached from Penitentiary Hill. Considering now only the embankments along the Colorado, it appears that the suggested treatment will require a railroad over their entire length, which is about 60 miles. These levees are already in existence, and the cost of equipping them with track (52-lb. rails) is estimated as follows:

Cost per mile of railroad on existing levee:

Steel, 52-lb. = 90 tons, at $30	$2 700
Fastenings	600
Ties, 2 600, at $1.25	3 250
Track laying	400
Train service	125
(Say, $7 000 per mile)	$7 075

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There is a railroad on the Reservation levee between Yuma and Laguna Dam, and deducting this length of 13 miles from the 60 miles required leaves 47 miles to be built. At \$7 000 per mile this cost would be \$329 000, to which may be added about \$10 000 for the Pilot Knob trestle, giving a total for railroad construction of, say, \$340 000. The average length of haul from these quarries will approximate 15 miles, and it should be done for not more than 1½ cents per ton-mile. The 1 500 000 cu. yd. will equal about 3 000 000 tons, and with average haul of 15 miles, gives 45 000 000 ton-miles, which at 1½ cents equals \$675 000. Using large steam shovels, quarries with long and high faces, mined with small tunnels in which heavy charges are used, it would appear that the material could be placed on the cars at an outside price of 30 cents per cu. yd.; to this should be added, say, 25 cents per cu. yd. for spreading and placing the rock on the slope, giving 55 cents per cu. yd. as the cost above that of hauling. At 55 cents, 1 500 000 cu. yd. would cost \$825 000, which, added to the expense of hauling, makes a total charge of \$1 500 000, which is increased to \$1 840 000 when the cost of building the railroad is included. Adding 10% for contingencies and repairs gives a round sum of \$2 000 000 for protecting 30 miles of river, or about \$66 000 per mile, equivalent to \$33 000 per mile of bank.

Although these estimates must be considered as approximations, it appears that permanent rock revetment can be had for not more than one in which mattresses are used, and probably for less. The permanence of the rock, its slight maintenance cost, the ample quantity available, and the fact that it can be placed as well during attack at high water as at low, makes it the best and most economic protection for the banks of the river between Laguna Dam and the Mexican boundary. Once the tracks were on the levee, exposed points could be quickly protected, and the remainder of the work could proceed as desired.

By Deflecting Currents.—Bank protection by deflecting the currents away from the shore so that erosion cannot occur may be done by spur-dikes or by opening channels across sand-bars which are the cause of the impingement. The deflection of currents by brush dikes has been tried in many localities and with varying degrees of success. They are subject to destruction by the natural course of deterioration, by deep scour which occurs around their outer ends, and by the undercutting of eddies below them which they have created. They have been tried sufficiently along the Lower Colorado under the writer's direction to demonstrate that at best they are only a temporary expedient. If built of cottonwood piling, which grows along the banks of the stream, their life would not exceed 2½ or 3 years as the maximum; probably the average is not more than 2 years. Constructed of fir piling, they could be depended on for 6 or 7 years. If, at the end of

Mr. Sellew. that time, the new banks which had been created by silting between them were not protected by revetment, it is probable that the dikes would need renewal, in order to hold the work. One other disadvantage of spur-dikes is that the proper location for them is demonstrated only by the varying attack of the river, and, when this attack comes on, the water is generally so deep and the currents so swift that the dikes cannot be built. It is then necessary to defend them as best one can until lower water, and perhaps, on the falling stage, the river changes its point of attack, so that dikes are needed in other places. In the writer's judgment, protection by spurs should be used only in an emergency; they have no place in any permanent form of bank protection on this river.

Dredging.—There remains the method of diverting the current by opening up channels across sand-bars. Wherever the banks have been attacked, the impingement has been caused directly by the formation of deposits on the opposite side of the stream. As these deposits build out toward the opposite bank, the flowing prism of the river is narrowed, the concentration of the water increases the velocity, the channel is correspondingly deepened, and, as a result, there is deep water and high velocity against a caving bank. Under such conditions, if it were possible to open up a channel of sufficient dimensions across this bar, and if the current could be induced to follow this new channel, the attack on the bank would be immediately relieved. In some instances, where the encroaching bar is short and other conditions are favorable, channels have been opened up and relief secured by a liberal use of dynamite. Such a method is impracticable in most cases along the river. It is seldom that a condition occurs that can be relieved by explosives.

The following is taken from *Occasional Papers No. 41,* page 44:

"The experience with the revetment work previously done had shown it to be inadequate for the purpose, and rapid changes of type were not made until there was developed the standard of the present day: A fascine subaqueous mattress with upper bank paving of stone. But as the revetment work had gradually been increased in strength, it had also increased in cost, until in the new standard type the cost had reached about $30 per linear foot of bank, or approximately $150 000 per mile.

"As the large cost of a systematic improvement of the whole river became evident, the Commission was led to investigate means for temporarily improving navigation during the low-water season. The principal obstructions met with are the sand bars built up during high stages, through which the river slowly cuts a channel as the flood recedes. If this process could be assisted and hastened, navigation would be improved thereby, and to this end experiments were made with dredges. These experiments, started in 1894, were successful and as a result the Commission was led to reconsider the whole subject of the river improvement, and as a result of this study the Com-

mission decided to adopt a new plan for future operations, namely, to depend for the relief of the immediate needs of navigation upon dredging, and to restrict the improvement by bank revetment to special localities, where local interests or necessities demanded it." Mr. Sellew.

(From page 46): "From all these facts together it became apparent several years ago—not all at once, but gradually, as facts accumulated and the deduction of experience was unfolded—that it was not possible by any or all of the methods which had been employed to accomplish such permanent improvement of the river from Cairo down as was necessary to meet the urgent demands of commerce within any reasonable time to come. It was this disagreeable but unavoidable conclusion from the work of preceding years, together with the recent and wonderful development of the hydraulic dredge, that led the Commission to undertake the experiments of dredging the bars of the river with the view to the temporary improvement of its low-water channel which has been detailed in recent reports of the Commission and to which more extended reference is made elsewhere in this report. Those experiments have proceeded favorably so far as to hold out a reasonable probability of entire success."

A suction dredge of large capacity and light draft which could be maneuvered in the Colorado during flood stages would be of value in opening channels across sand-bars. During low water the dredge could be used in creating a channel about midway between the levees at those points where attack from caving banks was imminent. The excavated material could be deposited in the vicinity of the levee through pipes carried on pontoons, thereby building the banks and making their alignment more regular.

It might appear at first thought that the quantity of material to be moved would require a dredge of unheard of proportions, and that the expense would be prohibitive, but such work appears to be well within the range of the hydraulic dredges as developed for work on the Mississippi River, the largest of which have a capacity of 2 000 cu. yd. per hour. About one-third of their time is lost in tie-ups for repairs during high water, maneuvering for position on bars, and in change of position, so that throughout the year the dredge would operate not more than 240 days, or 5 760 hours. Such operation of the dredge would result in removing about 11 000 000 cu. yd. of material.

Assuming a low-water discharge of 4 500 sec-ft., passing Yuma with a velocity of 3 ft. per sec., the area of the flowing prism would be 1 500 sq. ft. If this was excavated, with a depth of 5 ft. and a width of 300 ft., the 45 miles of such channel between Laguna Dam and the Sonora line would contain about 12 400 000 cu. yd. It is clear that no such quantity of excavation will ever be required along the lower river in any one season, therefore the dredge under discussion has ample capacity. The questions to be examined are: (1) its cost; (2) whether it can be designed of dimensions which will allow it to navigate the

Mr. Sellew. shallow and tortuous channels of the Colorado; and (3) whether it can be operated during floods for opening channels across the bars.

Referring to Document 50, 61st Congress, page 55, where is shown a memorandum of a project for maintaining by hydraulic dredging alone a navigable channel, 14 ft. deep and 500 ft. wide, between St. Louis and the mouth of the Ohio River: The estimates provide for 20 dredges, each with a capacity of 2 000 cu. yd. per hour, to cost, with all attendant floating plant, tools, auxiliaries, etc., \$6 000 000, or \$300 000 per dredge. The annual maintenance is stated as \$100 000 per dredge. In estimating the cost of a dredge, its life is here assumed as 20 years, and a sinking fund is provided for its renewal at the end of that time.

Estimated Annual Cost of Dredges.

Interest at 3% on \$300 000	\$9 000
Annual maintenance	100 000
Sinking fund	11 166
	\$120 166

The maintenance of the channel by dredging would require continuous work, year after year, no permanent results being obtained. In about 16 years the sum of the annual maintenance would equal the estimated cost of the rock revetment, and the maintenance, capitalized at 3%, gives an amount double that for which it appears a safe and permanent bank protection may be obtained. Whether or not a dredge can be built to satisfy the local conditions cannot be stated from the data at hand, but as such operations will cost double those necessary for permanent bank protection, further examination in this direction appears to be needless.

Conclusions.

The purpose of this discussion is to indicate in a general way the present conditions along the Lower Colorado, and to point out, from an analysis of the data here assembled, the future course which will probably be productive of the best results. It appears:

(1) That in their present state, the levees are at the mercy of the meanders of the stream, except throughout those portions where a railroad occupies the levee top;

(2) That checking the meanders of the stream by permeable dikes during low water and brush defences during high water are simply emergency works which accomplish no permanent good;

(3) That owing to the large amount of scour during freshets and the fill-back on the falling stage, bank protection by means of willow mattresses as constructed in other places is impractical, if not impossible;

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(4) That bank revetment, permanent in character and reasonable in cost, when compared with other methods, can be readily and quickly accomplished with rock placed from railroad trains operated on the levee tops and connecting with existing quarries which are within a reasonable distance;

(5) That further investigation is needed before it can be predicted, with any degree of accuracy, what effect the possible storage of flood water on the upper reaches will have on the flood discharge and the meandering tendency of the lower river;

(6) That it is doubtful if the amount of control obtained in this manner will eliminate the necessity of bank protection, and, even if such control could be obtained, a number of years must elapse before it can be made effective;

(7) In the meantime, existing works will continue to be menaced, therefore protective measures of greater or lesser magnitude must continue. In view of this fact, and the probability that some caving banks will always exist, the money spent for their protection should be for permanent revetment.

Nothing in the foregoing should be construed as opposing in any degree the creation of storage works at the head-waters of the Colorado. All feasible sites should be carefully examined, their probable influence estimated, and a well-determined construction plan matured. Such preliminary action is necessary before the advantages to come from the complete development of this river can be realized.

The destiny of this stream should be controlled by engineers rather than by statesmen, as the author suggests.

Mr. Mead.

ELWOOD MEAD, M. AM. SOC. C. E. (by letter).—The author's able review of the irrigation development in the Imperial Valley and the measures taken to control the Colorado River, describes the most unique achievement in American engineering since Eads deepened the mouth of the Mississippi. The Society is indebted to Mr. Cory for having given such a complete history.

If the gap in the Colorado had not been closed, Imperial and a score of other towns would have been buried deep under the waters of an inland sea; the grain fields and orchards of thousands of farmers, and representing the savings of lifetimes, would have been blotted out; one-twelfth of the irrigated area of California would have disappeared; the Laguna Dam would to-day have probably been only a memory; and the greatest change in the earth's geography which has taken place within recorded time would have been an established fact.

Only those who heard the turbulent torrent that roared through the break in the bank of the Colorado and saw the forest-covered lands along the new channel melt and disappear in its depths, could understand the appalling nature of the disaster which impended and the

Mr. Mead. tremendous odds which had to be faced to restore the river to its original channel.

In this attempt an expenditure of more than $1 000 000 had to be risked, with the chances on the side of failure. The courage with which Mr. Randolph and Mr. Cory assumed their trying responsibility, and the generalship which they showed in handling their immense equipment, are not made clear in Mr. Cory's paper. Only those who knew the conditions faced can fully appreciate the superb merit of their achievement.

The chief value of this paper, however, is not found in its description of the engineering methods used, but in the social and political conditions, attending irrigation development in the West, which it reveals. This history shows that, so far as this enterprise was concerned, there was an utter lack of public direction and supervision, or either State or National aid; and, as these are fundamental features of irrigation development, it is that portion of the paper which the writer proposes to discuss.

If it is the duty of the Government to conserve the public resources and to give any regard to the general welfare of the people developing them, this should be shown by the enactment of intelligent land and water laws and adequate administrative control of streams, as this is the only means of ensuring the full development of these resources and the right conditions of life in the arid West. This part of the country embraces two-fifths of the Nation's area. It was almost worthless without irrigation, it is immensely valuable now, but the change has been accompanied by a waste of effort, a loss of money, and the infliction of hardships and injustice which could have been easily prevented if the State and National law makers had done their duty.

When Western settlement reached the arid region, Congress should have provided for the diversion of rivers under public control, and in accordance with a general plan. This would have secured the irrigation of the best land, prevented the duplication of canals, or the building of canals where there was no water to fill them. It should either have built the large and costly irrigation works as Government projects, or safeguarded the private investments in these works, by making land and water security for the cost. Instead of this, there has never been any satisfactory title to water, and the public land has almost invariably passed into a control hostile to or independent of the irrigation works. Instead of adequate security, there was never, prior to the passage of the Carey Act and the Wyoming State law accepting this Act, any security for the money spent in irrigation works to water public lands, and at least three-fourths of these enterprises were financial disasters.

If all important irrigation works had been built by the Government, money could have been obtained at one-fourth the interest charge

the majority of private projects had to pay, and without the ruinous discounts on loans which the flotation of these enterprises entailed. Low interest rates would have made much lower water charges possible and thus removed from American irrigation its most serious handicap. Mr. Mead.

If there had been any attempt to frame a land law suited to requirements in the arid region, it would have at once been manifest that the homestead should have been reduced in acreage instead of increased, because it requires more labor to cultivate 80 irrigated acres than it does 160 acres watered by rain, and the returns are larger. Instead of reducing the homestead and restricting settlement to homesteads, however, the Desert and Land Act was passed, which gave 640 acres to the individual, conferring title without residence, without cultivation, and too often with the mere pretence of watering. The Desert Land Act was promoted by the rich range stock owner. It was the worst handicap irrigation ever had, and, instead of aiding the deserving settler, it took away his best opportunities. Instead of public control of streams, men fought for their possessions with shotguns and shovels along the banks, or with more costly weapons in the Courts, where litigation, instead of settling issues, as a rule, created new fields for controversy. The water-right litigation of the arid States has not only been a continuing source of ill-feeling, injustice, waste, and loss, but its existence is a reflection on the capacity of the law makers who are responsible for it. An understanding of these matters is necessary rightly to interpret what occurred in the Imperial Valley.

The irrigation and settlement of the desert of Southern California should never have been left to private enterprise. This was a case calling for direct Governmental control and responsibility. The desert was below sea level. It was worthless to the State, dangerous to travelers, and ugly to look upon. It was known that the Colorado River was a stream subject to torrential floods, and that the banks were alluvial; and it required only the slightest consideration to realize the danger which would menace settlers if they were allowed to make homes in this valley without the diversion works being made safe beyond question. The benefits which would come from irrigation and settlement made it a matter of direct concern to every resident of California, gave the project a National importance, and justified its construction as a public work.

If, however, the State did not build the works, it was its duty to supervise them and protect the private enterprise which did. Before investors were allowed to risk their capital or settlers risk their lives and comfort, some public authority should have seen to it that the land was fit to cultivate, and that there was a legal title to water; and there should have been an inspection of engineering plans and an investigation of financial arrangements.

Mr. Mead.

Instead of this, an irresponsible company without capital employs an engineer, whom it fails to pay. A right to 10 000 cu. ft. per sec. of a navigable international stream is established by the engineer posting a notice in an uninhabited jungle of weeds and brush. Of course, this was a farce, but it was the manner of establishing water titles prescribed by the law of California. That the title was worthless was shown later, but the early investors did not know this, they trusted the State to uphold and defend its statutes, and were betrayed. Any one who reviews this record must be amazed at the exhibit of imperfect laws and the incompetence or indifference of the public authorities. The surveys of public lands were shown to have been fraudulent, which was bad enough, but the delay of six years in correcting them was infinitely worse. The examination of the soil should have been made before settlement instead of afterward, and should never have been made except by experts having practical knowledge of irrigation. Instead of a proper land law, much of the land was acquired by non-resident speculators.

When the water title was attacked by the Interior Department, it was the duty of the State to defend its authority to grant titles and to protect this struggling project and the unfortunate settlers. For this protection, however, they had to look to a foreign country, and one wonders what would have been the result if Mexico had been as indifferent as the United States. This attack on the water title by the United States seems to have been morally wrong even if legally defensible.

Finally, when it was known that the river was beyond control, and the most appalling calamity that ever menaced the State of California was impending, it should not have been left to a bankrupt company and impoverished settlers to cope unaided with this disaster. It was the duty of the State or Nation to take charge, and provide the money and men needed to restore the river to its former channel. Apparently, no one in authority was interested, the State Government only considered the matter long enough to write a letter to the President, and the President, having Congress on his hands, shifted the responsibility to the head of a railroad company; and it was not until this railroad company took charge that we have the first refreshing example of generosity and public spirit. Nothing could have been finer than the action of Mr. Harriman. The loan of $250 000 when his time and resources were overtaxed by the earthquake at San Francisco, providing more than $1 000 000 for the last hazardous attempt to save the valley, furnishes an inspiring contrast to the supine indifference and irresponsibility shown by both the State and Federal authorities.

The Imperial Valley produced last year crops worth $10 000 000. The most desolate part of California has become attractive, productive,

and profitable; but, when one contemplates the hardship, anxiety, and suffering of settlers, and the waste and loss to the investors in the enterprise, one cannot help feeling that the conditions which imposed such burdens on development ought to be removed. Mr. Mead.

The National and State benefits which have resulted ought to have brought both wealth and public recognition to the pioneers of the enterprise; instead, its engineer and organizer was in the end cast on the scrap heap, his life work wasted; and a large percentage of the pioneer settlers, after untold hardships, had to give up and leave. Many of those now in the valley have had to pay high prices for their land as the result of early acquirement by speculators. The Southern Pacific Company, although requested by the President of the United States to save this valley from destruction, has not been paid for doing so. The speculators in the company, who were astute enough to get two dollars for one, have in many instances become rich; the lawyers who have had charge of the unending litigation have reaped a fruitful harvest; but those who worked for the real success of the enterprise, the men who risked their own comfort and that of their families to create homes, have paid penalties which are entirely unnecessary where irrigation development is carried out under a proper system of public control.

Mr. Cory's paper needs to be read in connection with that recently submitted by Mr. Lewis, the State Engineer of Oregon; both papers show the need for a change in the public conception of the duties and responsibilities of the Government in conserving and developing public resources, and a general appreciation of the need of more effective laws for safeguarding investments and making the settlement of arid lands easier, cheaper, and safer than this has been in the past.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A SUGGESTED IMPROVEMENT IN BUILDING WATER-BOUND MACADAM ROADS.

Discussion.*

By Messrs. F. G. Frink, C. H. Sweetser, and W. W. Crosby.

F. G. Frink, Assoc. M. Am. Soc. C. E. (by letter).—It is hoped that Mr. Meem will publish such information in regard to topography, climate, sub-grade, kind of stone, expected traffic, etc., as will show the range of adaptation of the design presented. In this way costly experiments may be avoided. Mr. Frink.

The writer's experience leads to the conclusion that the "variance in specifications," deplored by the author, is really one of the most encouraging features in the development of road building, as it implies a recognition of the widely differing conditions to be met. The use of a design, because it has been a success in some other locality, may result in loss of time and money.

For example, experience teaches that a road constructed with such a base as the one cited would probably fail in a section where, by porosity of the soil and seepage from higher land, the sub-grade was saturated for extended periods. In this case the base must be rendered as nearly impervious as possible by using fine material at the bottom to fill the voids.

Bearing in mind Macadam's insistence on the use of small fragments, the writer would suggest the query: Is a road ever the better for having large stone, say, greater than 1½ in., in any part of its section?

* This discussion (of the paper by J. L. Meem, Assoc. M. Am. Soc. C. E., published in December, 1912, *Proceedings*, and presented at the meeting of February 19th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Sweetser. C. H. SWEETSER, M. AM. SOC. C. E. (by letter).—The method of separating the screenings into two sizes, as suggested by Mr. Meem and used successfully on the Memphis-Bristol Highway, in Tennessee, seems to be an improvement over the usual specification for the application of the binder course. The writer has obtained a quicker bond by applying the screenings in small quantities until the required bond was obtained. When the dust of fracture is applied, together with the coarse material in the screenings, the dust when wet seems to "ball" up and prevent the coarse material from filling quickly the voids in the macadam. From the experience on the Memphis-Bristol Highway, the suggested improvement would seem to hasten the obtaining of a proper bond.

A few words on water-bound macadam and the automobile may not be out of place. Engineers should hesitate, and consider local conditions very carefully, before specifying the construction of water-bound macadam highways. With the improvement of the highway, automobile traffic is sure to increase, and it soon becomes necessary to construct a surface which will withstand this traffic. In the majority of cases the most economical time to do this is when the improvement is first made. Mr. Meem states that the suggested improvement is "less affected by automobile traffic." Why not construct at once, so that it will not be affected at all?

In spite of knowledge gained through the experience of road builders, both in the United States and Europe, there are miles of water-bound macadam roads being built in America where some other form of pavement should have been specified.

Mr. Crosby. W. W. CROSBY, M. AM. SOC. C. E. (by letter).—The author refers to the "bond" of the road, apparently meaning the cemented condition brought about by the action of water on the stone dust. If this deduction is correct, the writer cannot agree with one of the author's premises, namely, that such a "bond" is "the difficult and essential thing to obtain in a water-bound road."

If, however, the "bond" is, as the writer understands the term, the interlocking of the angular particles of stone with each other, supplemented by the cementing of the fine material, then the writer is prepared to agree with Mr. Meem to a certain extent. The qualification he has in mind is that such a bond, while always essential, is only difficult to secure with proper stone where the sub-grade for any reason is inclined to churn up into the macadam stone and by its intrusion prevent their proper compaction. Such a bond, of course, costs something to secure by rolling, but this does not seem to the writer to be a real "difficulty."

The method of construction proposed by Mr. Meem has been tried by the writer on various occasions, but without his becoming favorable to it. In one or two instances, where "granolithic" or "grit" (the

chips between the $\frac{1}{4}$- and $\frac{1}{2}$-in. screens) was in demand for other purposes, the crusher product was robbed of a part of this portion and the remainder was used as described by the author in two applications, without other benefit than securing in this way some "grit" for another use. Mr. Crosby.

It seems to the writer that probably the superiority of results, stated by Mr. Meem as secured by his method, comes from the application of the screenings in two relatively thin layers with thorough rolling between them.

Too many roads are built where, after rolling the second course (or "No. 2" stone), the screenings are applied heavily in one coat an inch or more thick, and rolled and watered until the surface seems to be cemented. Such roads seldom wear well, raveling shortly and continually. The cementing thus secured is superficial, and the macadam beneath is porous. If, however, the screenings are applied in thin layers, and each layer is flushed and rolled into the voids of the "No. 2's," then the macadam will be more dense and less likely to ravel. In the writer's opinion, with the latter method, it makes very little difference how the screenings then are made up as to sizes, unless they come from limestone. In the latter case a sufficient quantity of coarse particles or non-cementitious material must be added, by mixing or by separate application, to offset the extreme tendency of the fine limestone material, by forming a paste on the surface, to prevent the proper filling of the voids in the macadam.

The writer's specifications, in this particular, have been as follows:

"THIRD COURSE.

"*Material.*

"173.—The third course of the macadam construction shall consist of trap rock screenings varying in size from dust to 1-inch pieces. Other material than trap rock screenings may be used if approved by the Engineer. Limestone screenings shall not be used with a limestone second course, unless approved in writing by the Engineer, or unless clean sand, approved by the Engineer, is used in substitution under the direction of the Engineer, for at least 50% of the limestone screenings.

"*Spreading.*

"174.—After the second course of No. 2 stone has been rolled and completed as above described, the screenings shall be spread, but in no case shall screenings be used until the second course has been thoroughly rolled and compacted. The screenings shall be spread dry, with shovels from piles alongside the road or from dumping boards; but in no case shall the screenings be dumped directly on the second course. The quantity of screenings used shall be such as will just cover the second course.

Mr. Crosby.

"Watering and Rolling.

"175.—After the screenings are spread they shall be first rolled dry and then sprinkled with water from a properly constructed sprinkling cart, and then re-rolled with a steam roller weighing not less than ten tons. The amount of water necessary shall be determined by the Engineer. The rolling shall begin at the sides and continue until the surface is hard and smooth and shows no perceptible tracks from vehicles passing over it.

"176.—If, after rolling the screenings, the No. 2 stone appears at the surface, additional screenings shall be used in such places.

"177.—The rolling and watering shall continue until the water flushes to the surface. The rolling is to extend over the whole width of the road, including the shoulders."

The closing sentence of Paragraph 174 should be noted, and this, followed by Paragraph 176, illustrates the point above made.

The writer thinks he sees one danger in the method proposed by Mr. Meem. It is that, by first partly filling the voids in the surface of the "No. 2's" through the application of the grit and by then completely filling the remaining voids in the surface by the application of the finer sizes, the penetration of the finer sizes into the voids in the body of the macadam will be obstructed or prevented. It is the writer's experience that the best macadam will have a maximum density, not only on or near its surface, but also well down into the macadam itself.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

ON LONG-TIME TESTS OF PORTLAND CEMENT.

Discussion.*

By Messrs. Chandler Davis, L. J. Le Conte, and George G. Honness.

Chandler Davis, M. Am. Soc. C. E. (by letter).—While connected with the Dock Department of the City of New York, the writer carried on a series of long-time tests of Portland cements, covering a period of 10 years. The cements used conformed to all the requirements of the Department standard specifications of 1898, the principal clauses of which were as follows: Mr. Davis.

"Slow-setting cement, gauged neat, must not set within ½ hour after being mixed with water.

"The neat cement, after 7 days (1 day in air and 6 in water) must be capable of sustaining without rupture, a tensile strain of at least 300 lb. per sq. in.

"Mixed one part cement to two parts sand, it must, after 7 days (1 day in air and 6 in water), be capable of sustaining without rupture, a tensile strain of at least 125 lb. per sq. in.

"The neat cement, when set, must show no distortions or cracks.

"The time of setting will be determined by a wire $\frac{1}{24}$ in. in diameter, loaded so as to weigh 1 lb. The cement will be considered set when this wire is supported by the cement without showing any indentation.

"The tests for checking or cracking and for color will be made by moulding on thin plates of glass, three cakes of neat cement, 2 or 3 in. in diameter, and about ½ in. thick in the center, and with very thin edges. One of these cakes, when it is set perfectly hard, will be put in water and examined from day to day to see if it leaves the glass, becomes distorted, or if cracks show themselves on the thin edges. Such distortions or cracks indicate that the cement is unfit for use at that time.

"The second of these cakes will be allowed to set hard in moist air, at a temperature of about 200°, for about 3 hours. It will then

* This discussion (of the paper by I. Hiroi, M. Am. Soc. C. E., published in December, 1912, *Proceedings*, and presented at the meeting of February 19th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Davis. be placed in boiling water, and remain in boiling water for a period of from 24 to 48 hours, and, at the end of that time, the cake should still adhere to the glass, and should show no cracks nor distortions.

"The remaining cake will be kept in air, and its color observed, which for a good cement should be uniform throughout, and for a Portland cement, the color should be of a bluish-gray. This cake should also adhere to the glass and show no cracks or distortions.

"The standard sand will be clean, dry, crushed quartz, passing a No. 20 sieve and caught on a No. 30 sieve.

"Ordinary, fresh, clean water, having a temperature between 60 and 70° Fahr., will be used for the water mixture and immersion of samples."

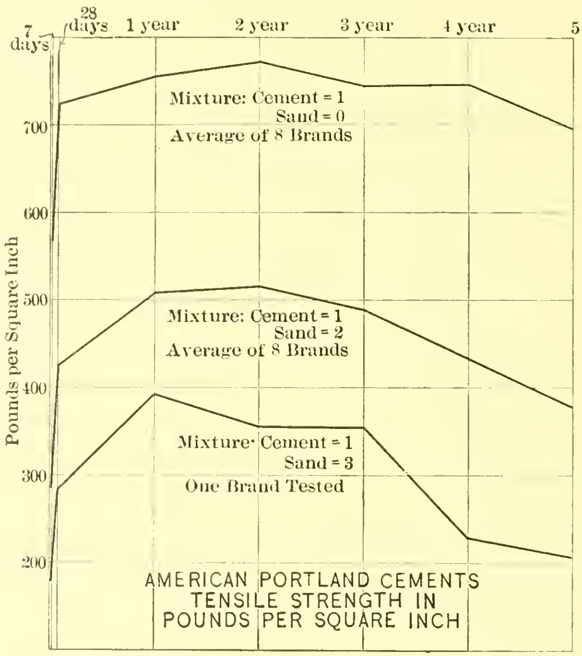

FIG. 4.

All the tests were for tensile strength, and the results obtained have been plotted on Figs. 4 and 5.

The curves shown on Fig. 4 are the average strength curves of several brands of American Portland cement. The resulting curves are very irregular. The cement seems to have acquired its maximum strength at the end of the second year and then gradually decreased in strength until, after a lapse of 5 years, the tensile strength is less than it was at the end of 28 days. The tests of the American brands were only continued for a period of 5 years.

The average results of seven brands of foreign cements are plotted on Fig. 5. Eliminating the 7- and 28-day tests, it will be seen that the neat cement reached its maximum strength at the end of 5 years, after having passed through a minimum at the end of 4 years. The strength of the cement at the end of the tenth year is only 17 lb. less than it was at the end of the fifth year. The strength curve for a mixture of one part cement and two parts sand, if the 7- and 28-day tests are eliminated, shows a maximum at the end of the first year, again a minimum at the end of the fourth year, with a continuation of the increase in strength through the fifth year, until the tenth year is reached, when it was only 37 lb. less than at the end of the first year. Unluckily, no samples were made for breaking between the fifth and tenth year, so no idea can be formed as to how the cement acted during that time. Mr. Davis.

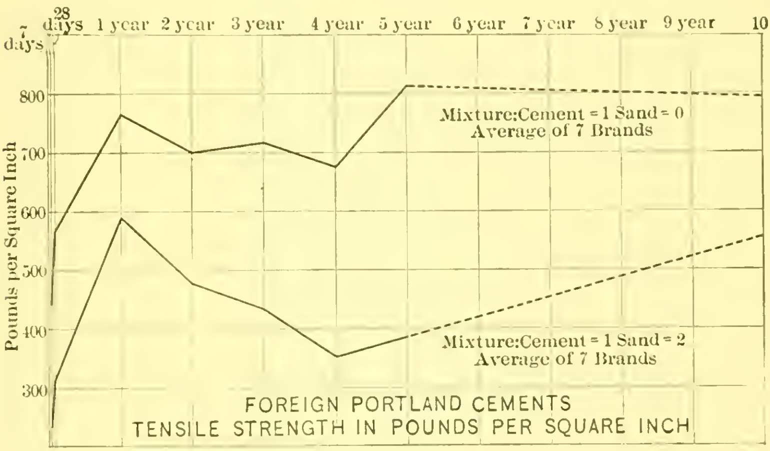

FIG. 5.

All the briquettes were moulded and broken by Mr. Robert M. Sterritt, who has been in charge of the Department Testing Laboratory since its organization, thus eliminating errors due to personal equation.

It is to be hoped that Mr. Hiroi will continue his series of very interesting tests, and will publish the results from time to time, simultaneously describing the condition of the structures in which these cements were used.

L. J. LE CONTE, M. AM. SOC. C. E. (by letter).—The writer is very much interested in this instructive paper. It is to be regretted, however, that the author does not state that all the briquettes were well protected against the hurtful effects of severe frosts in winter, though it is natural to presume that they were. Mr. Le Conte.

Mr. Le Conte. The experiments on sand briquettes are certainly very encouraging. The experiments on neat cement briquettes in sea-water, on the contrary, are very discouraging, from a theoretical point of view. In view of the fact that the great German cement expert, Dr. Michaelis, has clearly proved that the disintegration of Portland cement in sea-water is due to the chemical action of such water on the ordinary constituents of the cement, it is natural to suppose that some effect is to be expected, in the natural course of events—it being purely a question of time in any case.

Taking the worst case reported: If it can safely be assumed that the penetration of chemical action into the neat cement briquette is proportional to the loss in strength in a given time, then it amounts to approximately 0.32 in. in 4 years, and a tensile strength of 100 lb.; then, 10 years after, or 14 in all, we still have the same tensile strength, 100 lb., showing that chemical action had reached its extreme limit of penetration, namely, 0.32 in. There is certainly nothing very startling about this result, and, moreover, the concrete in all harbor works is necessarily exposed to marine growths of all kinds, such as barnacles, mussels, oysters, shell-fish, and all kinds of sea-weeds and grasses, which soon form a heavy coating over the entire water face of all the concrete work, and practically cut off all free circulation of sea-water. Hence the writer does not think that the interesting results reported furnish any information which calls for serious consideration; on the contrary, the sand briquettes stand up well under the 14-year tests, much better, in fact, than one would naturally expect, theoretically.

The probabilities are that the penetration of chemical action into the sand briquettes, which undoubtedly does exist, is so very small in amount as to be completely obliterated by the general slow growth in strength. The falling off in strength of neat cement briquettes between the ages of 5 and 14 years, when kept in ordinary air, is one of the mysteries yet to be explained. It seems to be the experience everywhere. It naturally follows from the above that low-grade porous concrete may have its strength seriously affected by the chemical action of sea-water.

Mr. Honness. George G. Honness, M. Am. Soc. C. E. (by letter).—Actual construction seldom affords opportunity to make long-time tests of cement, consequently, such information, though valuable, particularly in this age of concrete and the growing use of reinforced concrete, should be secured by tests made by manufacturers, laboratories of technical schools, or municipal departments having the direction of engineering works.

The construction of the New Croton Dam afforded an opportunity to make long-time tests, and the writer, though not connected with

that work, had access to the records, and believes them of sufficient value and interest to publish. Mr. Honness.

These records afford no opportunity to compare Mr. Hiroi's results in respect to conditions which obtain when the cement is used in salt water, but indicate the behavior of the cement and mortar under normal conditions. One brand of Portland and three brands of natural or Rosendale cement were used in this work, during the progress of which the methods of manufacture changed from the dome kiln to the rotary kiln. Practically all the testing was done by one man, which fact eliminates any inconsistency due to personal manipulation. The mortar briquettes were composed of 1 part cement to 2 parts standard sand, instead of the 1:3 mixture now prescribed; the standard sand was a crushed quartz, all of which would pass a No. 20 and be retained on a No. 30 sieve.

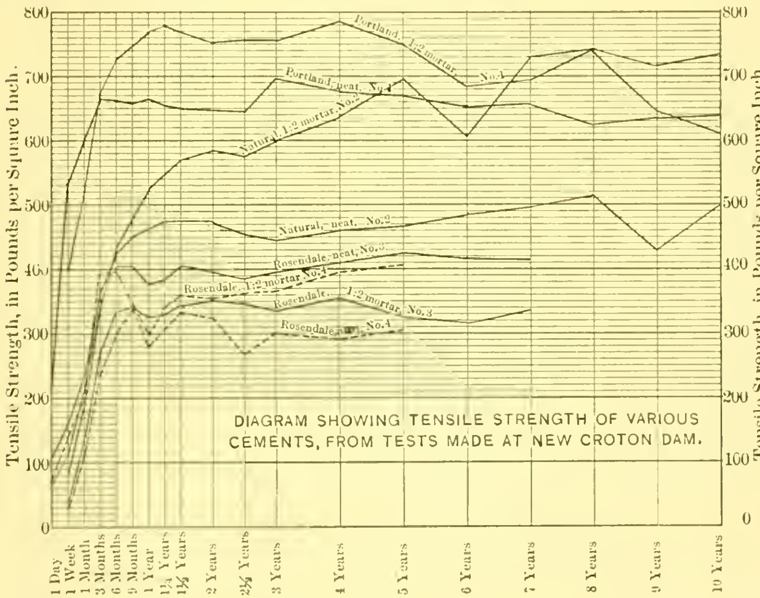

FIG. 6.

Table 2 shows the results of the tests, and it will be noted that they represent the average of a large number of briquettes. The diagram, Fig. 6, shows the results of the tests graphically. These tests show some interesting features, the chief of which is that the natural cement mortars show a gradual increase of strength to the end of the period, and the Portland cement shows retrogression of strength, after 3 years in neat cement and after 4 years in 1:2 mortar. Another interesting feature is that after 4½ years the natural 1:2 mortar exceeds the

Mr. Honness.

TABLE 2.—Tests of Portland and Natural Cements Used in New Croton Dam, 1894-1905.

Age.	Tensile Strength, in Pounds per Square Inch.															
	Portland, Neat, No. 1.		Portland, 1:2 Mortar, No. 1.		Natural, Neat, No. 2.		Natural, 1:2 Mortar, No. 2.		Rosendale, Neat, No. 3.		Rosendale, 1:2 Mortar, No. 3.		Rosendale, Neat, No. 4.		Rosendale, 1:2 Mortar, No. 4.	
	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.	No. of Briquettes.	Average.
1 Day..........	14 610	217	......	....	8 461	108	.....	....	12 265	64	.....	...	527	68	...	...
1 Week........	14 740	547	12 216	397	8 449	159	7 308	89	12 294	118	8 994	41	551	142	551	34
1 Month.......	1 985	601	1 938	529	1 000	235	979	190	1 079	220	1 070	139	85	192	80	114
3 Months......	1 398	669	1 378	677	815	361	785	342	565	374	580	270	75	392	75	232
6 Months......	1 018	664	1 011	727	665	424	667	441	480	403	510	333	56	393	58	298
9 Months......	832	658	797	748	628	450	624	489	395	402	390	342	40	342	46	341
1 Year.........	825	667	815	769	612	460	615	527	445	375	460	327	58	279	55	299
1¼ Years.. ...	686	654	671	779	570	471	558	550	420	381	400	330	37	301	32	344
1½ Years......	508	650	513	769	485	474	470	570	355	404	355	346	35	334	25	360
2 Years........	404	647	412	753	327	476	317	584	380	399	380	351	25	325	20	352
2½ Years......	367	646	384	755	352	453	335	575	325	386	300	348	25	267	20	362
3 Years........	370	697	335	755	277	443	255	600	290	394	275	335	15	300	15	363
4 Years........	197	674	205	784	183	459	190	635	235	409	185	356	15	290	15	398
5 Years........	164	669	171	751	150	466	155	697	220	423	232	328	20	304	20	406
6 Years........	27	651	90	684	22	482	35	604	105	416	90	319	...	...	...	...
7 Years........	95	658	73	695	61	494	83	729	55	414	54	337	...	...	...	...
8 Years........	71	624	85	739	79	516	104	745	......	...	.....	...	...	...	...	...
9 Years........	35	635	55	644	26	428	47	718	......	...	.....	...	...	...	...	...
10 Years........	33	639	18	611	50	500	49	732	......	...	.....	...	...	...	...	...

Rieble Bros. 1 000-lb. testing machine used.
Cement made by dome kiln method up to 1903; afterward by rotary kiln.
Briquettes up to 3 years broken by maker. Of the 4 to 10-year briquettes, 87% broken by maker.

strength of the Portland No. 1. This confirms the belief of some of the older engineers, that the lasting qualities of natural cement are superior to native Portlands. Mr. Honness.

The retrogression of strength shown by these tests, although it represents only one brand of Portland cement, makes one ask: Is this falling off of strength continuous, and when does it cease? This is of especial importance when considered in connection with reinforced concrete.

The author's experiments indicate that the brands of cement used by him continued to gain in strength after an age of 10 years. It may be that these brands have the proper combination of ingredients to assure a continual increase of strength, or at least no serious retrogression. Unfortunately, it is impossible for the writer to give the chemical composition of the Portland cement used at the New Croton Dam for comparison with the brands used by the author.

It seems desirable that, from data of this kind now available or to become available, a determination should be made of the chemical combination in cement which would assure that the strength would not fall below a fixed minimum after a long period of years. When the proper combination has been determined, it should be embodied in all specifications for important masonry construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TUFA CEMENT, AS MANUFACTURED AND USED ON THE LOS ANGELES AQUEDUCT.

Discussion.*

By Messrs. O. W. Peterson and Luigi Luiggi.

O. W. Peterson, Assoc. M. Am. Soc. C. E. (by letter).—The use of Haiwee tufa cement in the lining of about 40 miles of ditch of 930 sec-ft. capacity, during the past 3 years on the Olancha Division of the Los Angeles Aqueduct, has given the writer considerable first-hand experience with this product. This cement has been used, not only for lining the ditch, but in building various structures, such as arched flume crossings of 33 ft. span, etc. In fact, tufa cement has been used in exactly the same manner as any other cement, except that account has been taken of its slow setting. Mr. Peterson.

This open ditch section is excavated with long-boom, power shovels to as neat a line as possible, but, necessarily, in trimming the ditch to a true surface behind the shovels, back-filling is generally required. In such a dry country as this, which has an annual rainfall of only 5 in., the use of a well-compacted back-fill was considered, and has proven sufficiently good construction.

Against this trimmed earth section the 6-in. concrete lining is placed, without the use of forms. The finished ditch is 11 ft. wide at the bottom, with a 1-ft. invert, and the side slopes are 1 to 1. The depth of the ditch is about 13 ft. The concrete side lining is placed in alternate sections, 12 ft. wide, only 2 by 6-in. board strips being used to support the edges of the first set of alternate panels put in. Necessarily, the placing of concrete on such a steep slope without forms, cannot, even with the best efforts, result in the greatest compactness. Thus the tufa cement concrete is subjected to a further test, but again experience has shown that the strength of the slabs is suf-

* Continued from February, 1913, *Proceedings*.

Mr. Peterson. ficient to carry the load even against a support partly or entirely of back-fill.

The usual mixes of concrete have been 1 to 5 and 1 to 6, depending on the aggregates used. Generally, an extremely clean gravel, ranging from 40 to 75% of stone on the ¼-in. screen, has been easily obtained, but at places it has been necessary to add crushed rock.

The ditch is plastered with an extremely thin coat of 1 to 3 plaster, and for this purpose tufa cement seems to be especially well adapted, due largely to its fineness.

The temperatures in this locality range from 110° in summer to 8° in winter. A slow-hardening cement must be carefully protected at either of these extreme conditions. In summer, fresh work is sprinkled constantly for 2 weeks and covered with boards or canvas. In winter, boilers are used to furnish hot water for mixing purposes, thus turning out a warm concrete. This precaution, together with a covering of 1-in. boards or several layers of canvas, is sufficient protection against frost. The plaster is also mixed with hot water in winter, but generally no plastering is done in mid-winter.

From this brief description of operations, one can realize how difficult is the test to which the concrete for this work is subjected; and yet the practical completion of this division and the actual test of a full ditch of water for the greater part of 20 miles of work for 2 years' time has proven that the canal is water-tight. As the side concrete is really made up of separate 12-ft. slabs, slight joint lines show when the ditch is empty, but the expansion of the concrete due to moisture closes these, as well as other cracks, as the ditch is filled. Horizontal cracks are very infrequent, and arise from numerous causes, chief of which is probably occasional carelessness in back-filling. As a proof of the imperviousness of the lining, the leakage of the 20-mile section referred to has been less than a miner's inch per mile. This new ditch has never had running or muddy water in it.

The concrete has no tendency to swell under the influence of moisture, except to such an extent as all concrete expands in passing from a dry to a wet condition. This is a most desirable property for ditch lining, as it insures water-tightness.

In order to determine the value of cement rich in tufa, a carload of 75% tufa cement was obtained from Haiwee, and 300 ft. of concrete lining was put in with the usual mix. The laboratory tests showed that this special tufa was very slow setting. Frequent comparative tests for hardness were made of the special 75% tufa concrete and the ordinary 50% tufa concrete, using a pick for the purpose. At first the special tufa was much the softer, but as weeks went by the two products approached the same degree of hardness. At the last visit, when the age of both concretes was 10 weeks, there was but little difference in hardness. In fact, two visiting engineers were unable to detect any difference. This test seems to be proof of the

value of Haiwee tufa. There are no perceptible cracks or indications of disintegration in the 75% tufa cement concrete. It will be watched with a great deal of interest. Mr. Peterson.

In the lining of open canals, where proper care can be taken of the fresh concrete, slowness of hardening is no inconvenience. A saving of from 40 to 60% in the cost is sufficient consideration to warrant the extensive use of tufa cement. No doubt the use of long-boom, power shovels to excavate canals, and the lining of the section with tufa cement concrete could have wide application in the economical building of many large irrigation channels, resulting in a great saving of water now lost by seepage, and consequently a large extension of irrigated areas.

About twenty-five cars of straight Monolith cement have been used at times when the Haiwee mill could not supply the demand. Concrete made with this cement sets faster, but, as far as has been observed, it shows no advantage in ultimate hardness and strength over that made with Haiwee tufa.

A comparative test was made with blocks of tufa cement concrete in the strongly alkaline waters of Owens Lake and in fresh water. On removal, the concrete block which had been in the lake water 6 months was sound and apparently unaffected by the alkalis. It was not different in appearance from the block which had been in fresh water. However, this is not considered conclusive, as only from careful and extended tests can a thoroughly reliable conclusion be reached.

Luigi Luiggi, M. Am. Soc. C. E. (by letter).—From time immemorial, pozzolana, which is a variety of tufa, has been used in Italy for preparing mortar capable of setting under water. Mr. Luiggi.

The usual proportion is one volume of slaked lime and two volumes of pozzolana, mixed very intimately. Generally, the mixing is done by hand, but, lately, it has been done by machinery, which grinds the pozzolana more finely and mixes it more intimately with the lime. In this latter case the mortar is far superior, and sets more quickly, than when the mixing is done by hand. This is because the machinery makes a much more intimate mixture of the ingredients, and the silica of the pozzolana can act more directly and quickly on the hydrated lime.

All the harbor works by the Romans, which have lasted until now, for instance, that at Porto Trejano, at Porto d'Anzio near Rome, and Porto Adriano near Bari, were built with mortar, either for masonry or for concrete, made as indicated; and even now all, or nearly all, sea works in Italy are made in the same way, and always with excellent results.

However, pozzolana has one drawback: it sets very slowly, from 10 to 15 days in summer—when the heat accelerates the chemical reaction of pozzolana on lime—to from 15 to 30 days in winter. As it never freezes in Italian harbors, construction work is carried on throughout

Mr. Luiggi. the winter, but, during that season, the setting of the mortar is very slow.

For accelerating this setting, Portland cement, in the proportion of 100 to 200 kg. per cu. m. of pozzolana concrete, is often used. The setting then takes place in 24 hours, or even less, and thus the concrete stands better the washing of waves, when it is laid under water or is used for bag-work.

This concrete has always stood perfectly the chemical action of salt water, as the silica of the pozzolana neutralizes all the free lime, which is inevitable even in the best Portland cement. There is even an excess of silica, which makes the mortar rather acid—not basic—and thus the action of the sulphate of magnesia of the sea-water is better counteracted.

The necessity of having a mortar of greater resistance, and more rapid setting qualities, however, has induced Italian harbor engineers to adopt Portland cement mortar and concrete. The writer was perhaps the first to use it in 1882 in some parts of the Harbor of Genoa. In those days, as Portland cement was rather a novelty for Italian engineers, the mortar was made rather rich in cement, at least in the proportion of 1 cement to 2 of fairly graduated sand with from 30 to 35% of voids, and often even 1 to 1. These mortars, and the concretes made with them, have stood the action of salt water very well. On the other hand, with mortars made afterward, with a smaller proportion of cement, or with sand rather fine, that is, with more than 35% of voids, the results have been bad, and, in some cases, disastrous. This the writer attributes to the porosity of the mortar, which, under the shock of waves, or the action of tides, permits the salt water to penetrate and percolate through the mortar and decompose it. On the other hand, when the mortar is rich in cement, and as impermeable as it is possible to make it, the action of salt water is nil, or at least so much diminished as to cause no bad consequences.

Therefore, the writer always recommends his engineers to use at least 10% more Portland cement than the quantity necessary to fill all the voids of the sand, and the extra expenditure for cement is only a very small fraction of the cost of the mortar or of the concrete, and absolutely negligible in comparison with the damage that might result by the decomposition of the mortar.

There is always, however, the latent danger of the free lime, unavoidable in Portland cement, being acted on by the sulphate of magnesia, with the consequent decomposition of the mortar. It was for this reason that the writer, in 1888, used, and has recommended ever since,* the addition of some pozzolana in Portland cement mortars and concretes, in order to neutralize this free lime. In this he took advan-

* "La Pozzolana ed il Cemento Armato nelle Opere Marittime," *Giornale del Genio civile*, 1910.

tage of experience on the Roman sea-works. If they lasted some eighteen centuries, certainly the addition of pozzolana to Portland cement will correct the dangers of the free lime. Mr. Luiggi.

Experience in Italy since 1888 has been uniformly good, and now pozzolana is being used for improving the concrete for the construction of a very large graving dock in Venice, which will be 850 ft. long, 120 ft. wide at the entrance, with 40 ft. of water on the sill.

The composition of the concrete for this very important work is shown in Table 16.

TABLE 16.

Components for 1 cu. m. of concrete.	Concrete in contact with sea-water for a thickness of about 6 ft.	Concrete for the inner mass of the graving dock.
Portland cement, in kilogrammes....	400	250
Pozzolana, in liters....................	150	70
Sand, in liters.........................	450	440
Broken stone, in liters................	1 000	1 000

This really gives a little more than 1 cu. m. of concrete.

The practice of adding pozzolana, or its equivalent in trass, or tufa, to Portland cement for sea-works is becoming rather common. It is followed in Holland and Germany, and was adopted also by the Japanese in the construction of the Nagasaki Dry Dock, and, thus far, the results have been uniformly good.

Pozzolana, or trass, or tufa, are volcanic materials of very little cost, the transportation being the only real drawback to their more general use.

Besides neutralizing the free lime of Portland cement by the large quantity of silica in these materials, they tend also to make the concrete less permeable, by the colloidal action of the alumina they contain.

The usual proportion recommended by the writer is about $\frac{1}{2}$ to $\frac{1}{3}$ of pozzolana for 1 volume of cement, the higher proportion being for Roman pozzolana, containing from 51 to 53% of silica, and $\frac{1}{3}$ for Naples pozzolana, which contains from 62 to 65% of silica.

This addition of pozzolana, besides adding very little to the cost of Portland cement concrete, makes it less permeable and eliminates the danger of decomposition in sea-water. The experience on Roman works made with pozzolana concrete and exposed to salt water for more than eighteen centuries is a guaranty of the soundness of this practice.

Thus the use of tufa cement, or Portland cement reground with tufa, as practiced on the Los Angeles Aqueduct, is quite correct, both from the point of view of cost and durability of the work.

The mortar and the concrete will certainly be less permeable than if ordinary Portland cement had been used, and impermeability is an all-important condition in all aqueducts, especially in one of such magnitude as that of Los Angeles.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE INFILTRATION OF GROUND-WATER INTO SEWERS.

Discussion.*

By Messrs. John H. Gregory, Edward S. Rankin, Kenneth Allen, G. L. Christian, and E. Kuichling.

John H. Gregory, M, Am. Soc. C. E.—The author has presented a paper on a subject which is of especial interest to engineers who are called on to design, construct, or maintain, a system of sewers or a sewage disposal works. Leaky sewers are objectionable from many standpoints. It is not the intention of the speaker to mention all their objectionable features, but to call attention to one of the most important, that is, the consequent increased cost of construction, maintenance, and operation. Due to the infiltration of ground-water, not only must the sewers be larger than would otherwise be required, but also the sewage disposal works, if the sewage receives treatment before final disposal. Mr. Gregory.

The author has opened a way by which many data on the quantity of ground-water leakage into sewers can be obtained if each engineer having any data on the subject will contribute the same in the form of a brief discussion of this paper. The author will then have material which can be tabulated in concise form in his closing discussion, and should be of much service. The speaker believes that many engineers have some data on this subject, but hardly enough to justify them in writing a paper thereon, and, therefore, have not given the information to the Engineering Profession.

With reference to the author's headings under which such data should be tabulated, the speaker has one or two suggestions to make.

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* This discussion (of the paper by John N. Brooks, Jun. Am. Soc. C. E., published in December, 1912, *Proceedings*, and presented at the meeting of February 19th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Gregory. It would be desirable to include a column for the date—the year alone is sufficient. The engineer will then know how recent is the information he is examining. There should also be a statement of just how the sewer was constructed; if of sewer pipe, cement pipe, or reinforced concrete pipe, the lengths of the pipes, whether of 2, 2½, or 3-ft. sections, etc. Similarly, with monolithic, or other construction, data should be given as to both horizontal and cross-joints.

The speaker, however, is especially interested in the unit in which the leakage is to be reported. Perhaps the unit most commonly used is gallons per 24 hours per mile of sewer. This is very convenient, but it does not go quite far enough—it takes into account the length of the sewer, but not the diameter. The author presents a unit which has much to commend it, namely, gallons per 24 hours per square yard of inside surface. The unit is a rational one, but, unfortunately, is not a very convenient one to use.

The speaker suggests: gallons per 24 hours per inch of diameter of sewer per mile of sewer. A consideration of this unit will show that it takes into account not only length but diameter. He has used it to record the leakage of water pipes, which have been tested under his direction, and has found it extremely convenient. It also has the advantage that, in designing a system of sewers, calculations of ground-water leakage can be made very rapidly.

To show the application of this unit, Table 5 is presented. This has been computed from the author's data in Tables 1 and 2.

TABLE 5.—Infiltration of Ground-Water into Sewers.

Item No.	Place.	Diameter, in inches.	Length, in feet	Leakage, in gallons per 24 hours per mile of sewer.	Leakage, in gallons per 24 hours per inch of diameter per mile of sewer.
(1)	(2)	(3)	(4)	(5)	(6)
8	North Brookfield, Mass.	12	1 584	17 000	1 420
9	Rogers Park, Ill.	6	8 976	1 240	210
11	Altoona, Pa.	27	6 336	40 814	1 510
11	" "	30	3 168	86 592	2 890
11	" "	38⅝	5 016	264 000	6 830
16	....	3 ft. 8 to 6 ft. 0	47 744	7 280	123
17	Columbus, Ohio	42	1 744	6 340	151

Item No. 17 in Table 5, gives the result of a leakage test on a 3 ft. 6-in. by 3 ft. 6-in. concrete sewer of horse-shoe section, built under the speaker's direction at Columbus, Ohio, in 1906. The construction of the sewer was in charge of Mr. E. A. Kemmler, Resident Engineer, and the test was made under his immediate direction. This is the

speaker's humble contribution to the data which he would like to see tabulated in the author's closing discussion. Mr. Gregory.

It will be seen from Table 5 that the infiltration into the sewers listed varies from 123 to 6 830 gal. per inch of diameter per mile of sewer.

To illustrate the convenience of the unit suggested herein, assume that 300 gal. per 24 hours per inch of diameter per mile of sewer is a reasonable quantity (but the speaker does not care to go on record as stating that this is or is not a reasonable quantity). The leakage into a sewer, say, 18 in. in diameter, would then be 5 400 gal. per 24 hours per mile of sewer, a figure very quickly and easily obtained.

EDWARD S. RANKIN, M. AM. SOC. C. E.—Ten years ago it was the speaker's privilege to listen to a paper on this subject by Mr. A. P. Folwell, before the American Society of Municipal Improvements.* In that paper there was a table giving all the data Mr. Folwell was able to collect; it included gaugings on sixteen sewers. Mr. Brooks gives seventeen gaugings, of which six appear in Mr. Folwell's paper, the net total being twenty-seven. Mr. Rankin.

Considering only pipe sewers—to which alone this discussion will refer—and omitting those in which the leakage is only expressed in terms of population or percentage of capacity, neither of which units, as pointed out in the paper, has any direct bearing on the question, there remain but nineteen, and even these are so incomplete as to be practically useless in comparing the leakage of the several sewers mentioned.

The speaker is unable to agree with Mr. Brooks that the single unit of gallons per 100 ft. of joint would form an accurate basis for comparison. Although this includes the two factors of length and diameter, it omits the highly important factor of head of ground-water mentioned on the first page of his paper. The speaker would suggest that the true unit of comparison be expressed by the formula, $n\ d^x\ y\ h$, in which n represents the number of joints, d the diameter of the pipe, in inches, and h the head of water above the invert. From the data available, it would seem to be impossible at this time to determine with any degree of accuracy the values of x and y.

Mr. Brooks considers the leakage as directly proportional to the diameter. Alexander Potter, Assoc. M. Am. Soc. C. E., in his report on the Joint Outlet Sewer, states that, from his observation, the leakage varies as the square of the diameter. The speaker is of the opinion that the true value is less than unity, approximating to the square root of the diameter, and that the greater leakage in the larger pipes of a system is due to these pipes being in most cases laid at a lower level, and consequently being under a greater head of ground-water. He is

* "Perviousness of Sewers," *Proceedings*, Am. Soc. Mun. Impvts., 1903.

Mr. Rankin. unable to support this opinion by figures, but it is reasonable to suppose that the leakage is not uniform throughout the joint, but is confined largely to the weakest point, the few inches in the extreme bottom, most difficult to make, and practically impossible to inspect. If this theory is correct, the approximate value of the term, d^x, would become:

For an 8-in. pipe	$2\frac{3}{4}$ in.
12 " "	$3\frac{1}{2}$ "
18 " "	$4\frac{1}{4}$ "
24 " "	5 "

In regard to the other factor, h, there is abundant evidence to show its importance. In soil which is at all porous, every rain storm creates or increases the head, and the most casual inspection of the flow in most sanitary sewers will reveal a decided difference before and after a heavy rain.

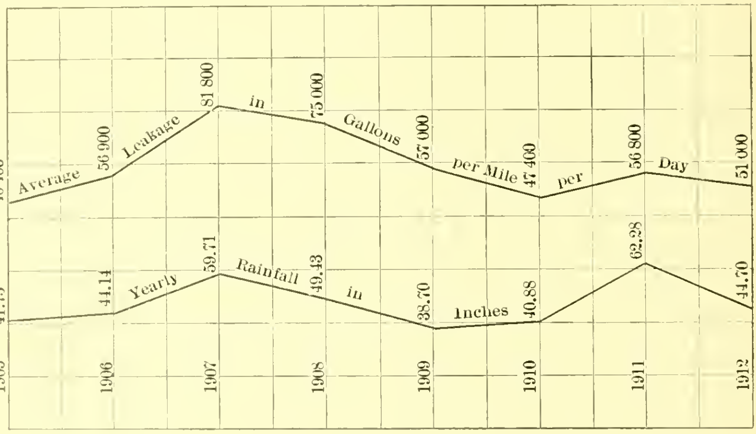

FIG. 1.

A gauging of about 4 miles of 8 to 15-in. pipe sewer in Newark, in March, when the ground was saturated with melting snow, showed a leakage of 38 000 gal. per mile, while in May only 13 500 gal. were found.

The diagram, Fig. 1, shows the total annual rainfall, in inches, and the estimated average daily leakage, in gallons per mile, on the Newark sections of the Joint Outlet Sewer for the past eight years. Automatic records are kept of the daily flow, and the leakage was obtained by deducting from the total a quantity equal to 500 gal. for each house connection. This is close to the actual quantity, as found in the dryest section of the sewer.

It will be noted that the curves follow each other, showing clearly the close relationship between leakage and ground-water. In the speak-

er's opinion, the co-efficient, y, would approximate unity, so that the formula would become $n\,h\,\sqrt{d}$. Mr. Rankin.

Although this formula is believed to be theoretically correct, and is useful in comparing the leakage in existing sewers, yet, for practical purposes in designing new work, the more simple form, expressed in gallons per mile or per acre, is to be preferred.

KENNETH ALLEN, M. AM. SOC. C. E.—In designing a system of sewers, it is often difficult for the engineer to ascertain what allowance should be made for ground-water. The speaker believes that it is high time that those who have the opportunity to note the conditions affecting the flow due to infiltration do this for the benefit of the Profession. Mr. Allen.

Data up to the present have been expressed generally in volumes per mile of sewer, or per capita daily, but such all-important elements as head of subsoil water, perviousness of the ground, and the size and character of the sewer itself, are either ignored or stated in such a general way as to be of little value.

There are those who believe that sewers laid in a porous soil, above the level of the ground-water, may be expected to lose an appreciable proportion of their contents by leakage, so that, instead of making an allowance for infiltration, one should more properly make a deduction in the estimated flow due to such leakage. The speaker believes such conditions to be exceptional, and that the ordinary pores left in well-made joints will gradually clog up; thus far, however, there appears to be no consensus of opinion on the subject.

It is desirable to forecast sewage volumes as closely as possible, in order to avoid spending money unnecessarily on sewers which will be too large, on the one hand, and from danger of damage due to backing up of sewage if the sewers are too small, on the other. In any case, excessive infiltration should be prevented, as it not only increases the first cost of the sewers, but, if the sewage is to be pumped or treated, it may increase the annual charges by a very large sum.

In Table 2 the author attempts to standardize the data relative to infiltration. This is a move in the right direction. The speaker would suggest, however, the addition of a column headed "Depth of Invert below Ground-Water Level," an important matter that seems to have been overlooked. The column, "Type of Section," should be omitted, as irrelevant.

No mention is made of one of the most prolific sources of infiltration, namely, the intersection of the sewer barrel with junctions, slants, and bulkheads. Moreover, house connections, if not laid with unusual care, are much more likely to contain imperfect joints, and even fractures, than the sewer itself, and their frequency, therefore, is an important consideration in estimating the quantity of ground-water to be expected.

Mr. Allen. During storms and thaws an appreciable quantity of surface water may be admitted through perforated manhole covers. In the case of the Joint Trunk Sewer of Essex and Union Counties, New Jersey, this was estimated to enter at the rate of about 1 600 gal. per manhole daily.*

With a system of well-constructed sewers, the initial infiltration is likely to decrease as the pores and crevices in the joints gradually become clogged by silt or deposits of salts, introduced by the ground-water itself or contained in the cement forming the joint. For example, in a pipe sewer recently laid in Batavia, N. Y., with a patent gasket composed of cement and sand placed in pockets of muslin dipped in glue, the infiltration at first was about 1 000 000 gal. per day in 31 miles of sewer, or 32 260 gal. per mile, with the ground-water standing from 3 to 5 ft. above the pipe; but later, after connecting 900 houses with the sewer, this rate fell to about 22 580 gal. per mile.†

The rates of infiltration in Table 6 are abstracted from discussions on the subject by George T. Hammond, M. Am. Soc. C. E., and the speaker.‡ and may be added to the list given by the author in Table 1.

TABLE 6.

Place.	Rate of Infiltration, in Gallons Per Day.	
	Per mile of sewer.	Per capita.
Alliance, Ohio. 15-in. pipe	195 000	....
Clinton, Mass	32 500	62.4
Concord, Mass. 18 % of sewers from 0 to 16 ft. below ground-water level	43 000	....
Framingham, Mass	31 180	65
Gardner, Mass	45 875	156
Madison, Wis. Trunk sewer. Dry weather	48 000	....
Malden, Mass. 60% of sewers in wet ground	50 000	65 to 70
Marlboro, Mass	59 540	144
Metropolitan (Boston) System, before making connections	40 000	....
Natick, Mass	77 420 to 100 000	209
New Orleans, La. Good construction. Wet soil.	31 800 to 60 000	....
Peoria, Ill. Vitrified pipe with brick sewers 15 gal. per sq. ft	100 000	....
Providence, R. I	....	47
Reading, Pa	5 172	....
Westboro, Mass. 15-in. sewer in wet ground	1 320 000	....
Worcester, Mass. 14 gal. per day per sq. ft. of interior brickwork	31 700 to 800 000	....

For Baltimore, Md., F. P. Stearns, Past-President, Am. Soc. C. E., recommended from 30 000 to 80 000 gal. per mile per day, but in the actual design of the system it is understood that a per capita rate—usually 20 gal. per day—was used.

It was the opinion of the late Freeman C. Coffin, M. Am. Soc. C. E.,

* *Engineering News*, February 11th, 1905.

† *Engineering Record*, November 9th, 1912.

‡ *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, pp. 690-707.

that, with 6-in. pipe, and with careful work, the infiltration could be kept down to 4 000 gal. per mile per day. Mr. Allen.

In the case of a large system of sewerage, it is often more convenient, for general purposes, to assume the daily volume of infiltration per square mile. In the preliminary studies for the sewerage of Baltimore, the speaker used, as suggested by the Consulting Engineers, Messrs. Rudolph Hering and Samuel M. Gray, Members, Am. Soc. C. E., the quantity, 100 000 gal. per sq. mile, increased to 750 000 gal. in the low areas bordering the water front.

More recently, in some studies for the Metropolitan Sewerage Commission of New York, the capacity of sewers has been estimated to provide for 450 000 gal. per sq. mile daily in partly developed districts and 1 000 000 gal. per sq. mile in completely built up territory. Assuming 15 miles of sewer per sq. mile in the former case and 25 in the latter gives: for suburban areas, 30 000 gal., and for urban areas 40 000 gal., per mile of sewer.

Assumptions or results in other cities are shown in Table 7.

TABLE 7.

	Infiltration. in gallons per day per square mile.
Louisville, Ky	1 250 000
New Orleans. La	1 250 000
Peoria, Ill (observed)	50 000 to 100 000
San Francisco, Cal. (pipe sewers)	55 300
San Francisco, Cal. (brick sewers)	166 000
Union and Essex Cos., N. J. (observed)	350 000
Washington. D. C. (very wet weather)	620 000
Worcester, Mass	450 000

The quantity of infiltration, especially for poorly laid brick sewers in a pervious, saturated soil, may be very great. The speaker recalls one case where the water spurted up through the invert of such a sewer in a number of places in jets at least the size of a pencil. On the other hand, concrete sewers may be made practically impervious by the proper proportioning and placing of the ingredients.

Although such sewers will be practically impervious under ordinary heads, it has been thought safer, in some cases, to resort to waterproofing. The first 3 590 ft. of the West Low-Level Interceptor at Baltimore was constructed of concrete with a 4-in. lining of brick. Between the concrete and the brick lining was placed membrane waterproofing consisting of two layers of 10-oz. burlap saturated with asphalt. The sewer varied from 74 to 84 in. in diameter, was provided with numerous house connections aggregating 2 700 ft. in length and was bulkheaded with timber at the upper end. It was laid on a marginal street along the water front with the invert from 11 to 13 ft. below tide, and included a section of 100 ft. passing just below the bed of a tidal stream. When completed, the infiltration amounted to

Mr. Allen. about 16 000 gal., or 0.223 gal. per sq. ft. of interior surface, per day, probably half of which leaked in around the bulkhead.

With pipe sewers, the only trouble, except with fractures, lies at the joint, and in wet ground such joints should be made with special care. In yielding ground, there should be a special foundation, unless the joint is made of some substance, like bitumen, that will remain to a certain degree plastic. For this purpose, the speaker has used cement, kneaded with tar pitch, with success. About ¼ mile of from 8 to 20-in. pipe laid in this way was tested by filling with water and found practically impervious under a head varying from 3 to 4 ft.

In hard ground, a mixture of sulphur and sand, or "leadite," may be preferred.

Whatever material is used should not be subject to deterioration or be either so plastic as to flow from the joint or so rigid as to crack under the conditions in which it is placed.

The entire subject is one involving so many factors, some of them obscure, that probably nothing closer than a fair approximation for considerable lengths of sewer can ever be hoped for; and, for short lengths, a larger margin for error must be allowed. With a knowledge of the local conditions, however, and more complete data regarding existing sewers, the engineer should be able to avoid any serious error of design due to this cause.

Mr. Christian. G. L. CHRISTIAN, M. AM. SOC. C. E.—The infiltration of ground-water into a sewer depends on so many variable conditions, none of which is exactly similar in any two sewers, that it would seem to be quite impossible to establish a standard of construction from which to determine definitely the degree of impermeability of any two sewers constructed in exactly similar ground, the depth of the ground-water being the same in both cases. It also makes a great difference whether the sewer is a sanitary one, pure and simple, or is part of a combined system. If the former, it is usually desirable to reduce the infiltration to a minimum, the joints being generally tightly caulked with oakum, then filled with cement mortar, and wound with cheese-cloth to hold the mortar in position until it is well set; spurs with loose covers are the exception, rather than the rule. In a combined system, the ends sought are, in a measure, quite different, part of its province being to lower the ground-water level permanently; the joints in pipe sewers are not caulked, being filled with mortar only. For many years it was the custom in the Borough of The Bronx, New York City, to put in a spur on all sewers every 3 ft., except at block intersections. Some years ago this practice was changed, and the distance is now 6 ft. These spurs have loose vitrified covers, and are capable of lowering the ground-water level permanently, regardless of the impermeability of the concrete or brick, or of the joints in pipe sewers.

The design of a combined system of sewers is based, primarily, on the intensity and duration of the maximum rain storm, or on the

average of a series of heavy storms, in a given locality, and it is evident, therefore, that the quantity of ground-water infiltration in such a design is so small that it becomes negligible. As to the skill in doing the work, cementing the joints, mixing and placing the concrete, and the character of the aggregate, etc., if one is to depend on the contractor's opinion, it will be first-class in every particular. On the other hand, if the decision is left to the engineers in charge of the work, there are apt to be as many opinions as there are engineers. The character of the trench, as to whether it is wet or dry, is also a relative term, and is apt to be colored by local conditions, that is, what would be termed a wet trench in a region where ground-water is generally high, might mean something quite different if recorded by an engineer doing work in a region where the ground-water seldom appears in the trenches. Mr. Christian.

A little more than ten years ago, the Sewerage and Water Board of New Orleans, La., made some careful observations, from which it was found that the actual infiltration into the sewers practically checked previous assumptions—that the quantity of ground-water to be provided for would be equal to 0.003 cu. ft. per sec. per acre, which is equal to 1 250 000 gal. per sq. mile in 24 hours. It might be said, in passing, that the ground-water level in New Orleans is very high, and all the sewers are below it. On the foregoing basis, there would be 32 miles of streets in 1 sq. mile, assuming it to be laid out in blocks 400 ft. long and 200 ft. wide, with streets uniformly 60 ft. wide. The infiltration, therefore, would average 39 000 gal. per mile per 24 hours. This figure is also used by the Borough of Brooklyn, New York City; in the Borough of Queens, the assumption is 75 000 gal. per mile per day. Since these observations were made, New Orleans has built some hundreds of miles of pipe sewers, and as all, or substantially all, of its sewage has to be pumped, it is possible to measure quite accurately the quantity of infiltration. It was the speaker's good fortune to witness the laying of some of the pipe, and he noted that the utmost care was taken to make the work water-tight. It was also noted that spurs were put in on each side, every 30 ft., opposite vacant property, the covers being tightly cemented inside the bell. Notwithstanding the care used, there is still considerable infiltration, the actual quantity being shown in the following extract:

GROUND-WATER FLOW, STATION A.*

Year beginning.	Ground-water per mile of sewer, per day, in gallons.
1907	55 000
1908	53 000
1909	51 000
1910	51 000
1911	48 000
1912	42 000

*Twenty-fourth Semi-annual Report of the Sewerage and Water Board of New Orleans, La., p. 100.

Mr. Christian. The author describes a concrete sewer in which the infiltration amounted to 0.8 gal. per day per sq. yd. of interior surface, which amounts to 7 300 gal. per mile per 24 hours. From the speaker's experience with the construction of concrete sewers, he would regard this as rather high, unless there was some leakage at the joints, or where each day's work joins that previously laid; and he suggests that a careful inspection would find that to be true.

In conclusion, it may be said that the design of a system of sanitary sewers in any town is a law unto itself; and, in designing such a system, it is wise to consider that the ground-water may have to be taken care of whether one wants to or not. Indeed, in the early stages of such a system, with only a few connections, and many of the sewered streets sparsely populated, it is often desirable to have many unconnected spurs through which water may enter, in order to assist in flushing the sewer. As the district is built up and the houses one by one are connected with these spurs, the need for the ground-water will cease, the house sewage taking its place. Coincident with the building up of the district, it might be assumed that the paving of streets and of court yards is taking place, which would probably mean a corresponding lowering of the ground-water and a reduction of infiltration into the sewer.

Mr. Kuichling. E. Kuichling, M. Am. Soc. C. E.—A number of useful suggestions, in regard to the form in which data pertaining to infiltration should be presented, have been made by the preceding speakers, so that little further remains to be said. The principal factors are the character of the soil, the extent to which it is saturated with ground-water, the elevation of the surface of this water above the top of the sewer, and the manner in which the sewer itself is built. An extensive stratum of sandy or gravelly soil often contains and yields a large volume of water, whereas little is delivered from clayey deposits; and in rocky strata seams are frequently encountered which deliver water very copiously. The quantity of water in the subsoil also varies with the daily and seasonal rainfall and the magnitude of the absorptive territory exposed thereto. It is obvious that much more ground-water will be found in the sandy soil at the bottom of a broad valley than in a similar soil on a narrow ridge.

One of the benefits usually claimed as resulting from a comprehensive system of sewers in a community is the permanent lowering of the existing ground-water table to the level of the sewers, thereby attaining dry cellars automatically and much less dampness of the air in dwellings. Observation has proved that this claim is well founded, and that in most cases shallow wells, in premises adjacent to streets provided with comparatively deep sewers, become permanently dry, even when the utmost care has been taken to make the sewers water-tight.

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Various reasons for this occurrence have been given in the case of sewers not equipped with under-drains. A common explanation is that the ground-water escapes by flowing along the smooth exterior surface of the main and lateral sewers and finally reaches the outfall; another one is that much less care is given to the construction of the numerous small lateral sewers, whereby the ground-water is enabled to enter through many defective joints and thus find its way into the well-built main sewer; and a third one is that, in the course of a year or two, many small cracks will develop in the entire sewer system, in consequence of settlements and unavoidable changes of temperature, thereby affording the ground-water access to the interior. Each of these explanations is satisfactory in some measure, but it is more likely that all the causes mentioned combine to accomplish the result.

In former years it was a frequent practice to build sewers with open joints at the bottom to admit the ground-water. An interesting measure of the quantity of such infiltration into an extensive sewer system built in this manner is given by Ernest Adam, Engineer of the Department of Streets, of Newark, N. J., in the discussion of a paper on the subject before the American Society of Municipal Improvements, on October 21st, 1903. He stated that many of the old brick sewers of Newark were made without mortar joints in the bottom, the result being a sewage flow of 48 000 000 gal. per day though the water supply of the city was only 25 000 000 gal. per day. It may be remarked that in 1903 the population of Newark was about 268 500, and that on December 31st, 1903, the total length of all public and private sewers in that city was 198.7 miles.* The total area of Newark was then 14 976 acres, of which 10 679 acres was dry land surface, the remaining 4 297 acres being tidal meadows and water.† From these figures, it will be found that the stated daily infiltration of 23 000 000 gal. of ground-water into the sewer system of Newark corresponds to average rates of 85.7 gal. per capita, 115 752 gal. per mile of sewer, and 2 154 gal. per acre, or 1 378 401 gal. per sq. mile of dry land surface, per day.

It is reasonable to consider that the soil under and adjacent to inhabited buildings will eventually be drained in some manner, either directly by special systems of tiles, or indirectly by defective joints in the lateral sewer pipes; and it is also fair to assume that in most cases the ground-water thus intercepted will be discharged into the public sewers. The total area of the sewered territory, therefore, becomes a factor that must be taken into account, together with the adjacent uplands which may contribute to the flow of ground-water in the populous districts. In some cases much of the flow from the uplands is intercepted by the highest line of sewer, thereby greatly relieving the satura-

* Annual Report, Board of Street and Water Commissioners, Newark, N. J., 1903, p. 118.
† U. S. Census, Report on Cities, 1905, p. 111.

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tion of the lower ground; but in most instances the sewers are built in streets which cross the contours more or less at right angles, so that ample space is left between such streets for much of the upland ground-water to flow down to the lower districts and seek an entrance into the sewers there.

Some notion of the quantity of flowing ground-water per unit of tributary area may be gained from measurements of the yield of natural springs which have a well-defined water-shed. Numerous observations of this kind in central Switzerland have shown that the normal minimum yield of such springs is 154 gal. per acre of tributary water-shed per day, when derived from the fine-grained sandstone called "molasse"; from four to six times that quantity when derived from moraines; and from eight to ten times that quantity when derived from the gravelly soils of valleys. Similar observations in Germany* indicate a minimum yield ranging from 154 to 924 gal. per acre of tributary water-shed per day; an average yield ranging from 462 to 1 232 gal.; and a maximum yield ranging from 770 to 4 620 gal. and upward, depending on the nature of the subsoil and the distance traversed by the percolating water. These yields from flowing springs may also be compared with the observed dry-season flow of rivers. In the New England and Middle States, the least monthly flow of streams having a drainage area of more than 100 sq. miles usually ranges from 0.10 to 0.25 cu. ft. per sq. mile per sec., which is equivalent to a yield of from 101 to 253 gal. per acre per day. This quantity is less than the aforesaid minimum yield of springs, but it must be remembered that in a considerable portion of a large water-shed the soil is generally unfavorable for absorption and percolation.

A better estimate of the quantity of ground-water that may reach a system of sewers is afforded by the results of numerous land-drainage experiments in England, France, Germany, and the United States. It was found that the quantity of rainfall which would be absorbed by the soil and percolate to the under-drains varied greatly with the character of the soil, the nature of its surface with regard to plant growth, the season of the year, and the magnitude of the rainfall during one or more days. A few references to such data may be of interest. J. Bailey Denton, a well-known English authority, states† that the maximum discharge measured by him from the under-drains of a large tract of free-soil land at Hinxworth took place in January, and was at the average rate of 0.16 cu. ft. per acre per min., which corresponds to 1 710 U. S. gal. per acre per day; and Charnock's observations, of the discharge of field under-drains at Holmfield during 1842-46, exhibit a maximum rate of 0.24 cu. ft. per acre per min.,

* Keilhack. "Grundwasser und Quellenkunde," Berlin, 1912, p. 447.

† "Sanitary Engineering," London, 1883 (?), p. 53.

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or 2 585 gal. per acre per day. French authorities, like Hervé-Mangon and Delacroix, state that the system of under-drains should have a capacity to remove a depth of at least 0.24 in. of water per day on the entire area, which corresponds to 6 517 gal. per acre per day. In Germany, the Silesian Agricultural Commission recommended that the under-drains should remove not less than 2 845 gal. per acre per day from low-lying flat areas, and 3 390 gal. from higher flat areas. Other German authorities on land drainage, like Vincent and Perels, advise the removal of 6 983 gal. per acre per day.

In the United States the subject of land drainage has recently been investigated by the Department of Agriculture, and in the report of such observations in Illinois and Iowa, published in 1908,* it is stated that the tile under-drains of areas ranging from 400 to 1 040 acres of black open loam with a clay subsoil frequently gave a maximum discharge of 0.16 in. of depth in 24 hours, or 4 345 gal. per acre per day; also, that in some cases the under-drains were engorged, and that it would be better to remove a depth of 0.25 in. of water in 24 hours, or 6 788 gal. per acre per day. It may be noted that the lines of lateral drains were here from 100 to 250 ft. apart, and it was recommended that in heavier or more clayey soils they should be placed from 40 to 80 ft. apart.

The foregoing discharges from the under-drains of agricultural land were all carefully measured, and can be regarded as a good indication of the quantity of ground-water to be expected in the outskirts of a city. The figures range from 1 700 to 7 000 gal. per acre per day, or from 1 088 000 to 4 480 000 gal. per sq. mile per day, and relate to surface strata of ground from 3 to 4 ft. thick. It is very probable that much larger quantities would be found at times in strata 8 ft. or more in thickness, corresponding to sewered territory. Larger figures would also be obtained by using the results of the numerous percolation experiments which have been made with lysimeters of small area. The data submitted, however, suffice to show that much water is present in the subsoil after a period of heavy rainfall, and that it will enter the sewers under considerable head if opportunity is offered, as in the aforesaid case of the Newark sewers.

Two very instructive measurements of the infiltration into a system of earthenware pipe sewers were made at Stamford, Conn., a number of years ago. An aggregate of 13.38 miles of such sewers, ranging from 6 to 18 in. in diameter, had been laid in 1888 under the direction of the late Col. George E. Waring, Jr., in a wet gravelly soil under an area of 746 acres, or 1.16 sq. miles, of that city, and the infiltration was measured in May, 1889, when only 215 house connections had been made. The quantity was found to be 885 gal. per acre per day, or

* Cited also in "Engineering for Land Drainage," by Charles G. Elliot, M. Am. Soc. C. E., Chief of Drainage Investigations, U. S. Dept. of Agriculture, Second Edition, New York, 1912.

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566 400 gal. per sq. mile per day, or 49 340 gal. per mile of pipe per day, on the average. In September, 1892, when 530 house connections had been made, it was measured again by Col. Waring and John Bogart, M. Am. Soc. C. E., and was found to have increased to 1 689 gal. per acre, or 1 080 960 gal. per sq. mile, or 94 170 gal. per mile of pipe per day.* The reason for this large increase is doubtless that numerous joints had become defective, in the course of three years, and thus facilitated the admission of the ground-water.

Numerous other data, not given in Table 1, will be found in two paper† on the subject by Mr. A. P. Folwell and E. D. Rich, M. Am. Soc. C. E. In Mr. Folwell's paper it is stated that in many sewer systems the infiltration is from 50 to 100% of the volume of house wastes; also that in Boston, Mass., Howard A. Carson, M. Am. Soc. C. E., estimated, from extensive measurements of the flow in the main sewerage system in 1889, that the infiltration was at the rate of 45 gal. per head of population per day, and in Malden, Mass., and Providence, R. I., it was, respectively, 65 and 47 gal. per head per day. It is also stated in the same paper that, in a length of about 4 000 ft. of brick sewer in East Orange, N. J., the rate of infiltration was 570 000 gal. per mile per day, while in 21 miles of 8-in. pipe sewer it was only about 9 000 gal. per mile per day; that in Malden, Mass., it was 50 000 gal. per mile per day on the average for 38 miles of sewer, and that, if only the length laid in wet ground were taken into account, it would be 83 000 gal. per mile per day; that in Westboro, Mass., in 1899, the infiltration into 1 950 ft. of 15-in. vitrified pipe sewer was 489 000 gal. per day, or at the rate of 1 320 300 gal. per mile per day, and in another section of the same sewer, 1 060 ft. long, it was at the rate of 610 000 gal. per mile per day; that in New York City the infiltration into a system of 5 miles of sewers, ranging from 8 to 24 in. in diameter, was found to be about 1 300 000 gal. per mile per day; and that, according to a report made in 1900 by X. H. Goodnough, M. Am. Soc. C. E., to the State Board of Health of Massachusetts, about 137 miles of sewers, ranging from 8 to 36 in. in diameter, had recently been constructed in and near Boston, in which the infiltration was at the average rate of 40 000 gal. per mile per day before any buildings had been connected thereto.

In Professor Rich's paper it is stated that in a 20-in. pipe sewer in Canton, Ohio, the infiltration was at the rate of 70 000 gal. per mile per day; and that in a 12-in. pipe sewer in New Bedford, Mass., it was 17 000 gal. per mile per day. These two papers also refer to some of the other data given in Table 1. In describing the sewage works of Fond du Lac, Wis.,‡ George S. Pierson, M. Am. Soc. C. E., stated

* *Engineering Record*, 1892, II, p. 247.

† *Engineering Record*, November 7th, 1903, p. 564, and October 1st, 1910, p. 377, respectively.

‡ *Engineering News*, May 22d, 1902, p. 411.

that the soil in which the sewers were laid was a very retentive red clay, and that the infiltration into the new 24-in. vitrified-pipe outlet sewer was carefully measured before the old sewers were connected thereto. This new sewer had been laid below the level of low water in the river, and was a little more than 7 000 ft. long. The infiltration was found to be less than 3.0 cu. ft. per min., and was regarded as very slight. From these figures we obtain in said 24-in. pipe a rate of 24 370 gal. per mile per day. Mr. Kuichling.

In Table 2, Item No. 16, the author gives the average rate of infiltration into a reinforced concrete trunk sewer, 47 744 ft. long and ranging from 3.67 to 6.00 ft. in diameter, as being 0.8 gal. per day per sq. yd. of inside surface. This information is interesting, but it should be supplemented by naming the locality and describing the nature of the subsoil along the route. It would also have been useful if the author had stated explicitly that the aggregate interior surface of that sewer was 82 123 sq. yd., and that the measured infiltration was about 65 700 gal. per day, or at an average rate of 7 266 gal. per mile per day, which he leaves his readers to ascertain. In comparison with the other observations cited herein, this rate of infiltration is extremely low, and indicates either great water-tightness of the concrete, or an unusually dry subsoil at the time of measurement.

Reference was made in the foregoing to the probable increase in the quantity of infiltration in the course of time. This is to be expected, as small settlements of a sewer usually occur in soft ground, whereby cracks are produced. In the case of large pipes, such cracks are often due to the rapid settlement of the back-fill after a thorough saturation with water. The speaker has seen many miles of pipe sewer laid truly to line and grade originally, and has found them after ten or more years showing numerous irregularities of both line and grade, as well as many fractures. The same is also true of large brick and concrete sewers. In structures of this kind in wet ground, few have ever been built that did not exhibit considerable infiltration in the outset, and it is not likely that they became tighter subsequently if the external head of water remained unchanged. Cracks are likewise formed by the unavoidable contraction of the material when the temperature is lowered for a long period. It must be presumed, therefore, that the infiltration into old sewers will be greater than into new ones, other external conditions remaining the same.

The subject is important, notwithstanding that it has been treated very briefly in most textbooks on sewerage. It becomes specially prominent in a separate system when the sewage has to be pumped or treated in disposal works, and much of the initial capacity of the pipes is taken up by infiltering ground-water. It is hoped, therefore, that the author will succeed in eliciting and arranging in convenient

Mr. Kuichling.

form many additional trustworthy data. The proposition to estimate the infiltration by the square yard of interior surface is good, but it will probably be more convenient to express it in terms of per foot of diameter and a length of 100 ft., or even 1 000 ft. In any event, the nature of the subsoil and the height of the ground-water surface above the top of the sewer should be given, together with the date of the observation, and a brief description of the construction, and its age.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

HYDROLOGY OF THE PANAMA CANAL.

Discussion.*

By Henry L. Abbot, Esq.

Henry L. Abbot, Esq.† (by letter).—Mr. Saville has made an interesting and valuable contribution to the technical literature of the Panama Canal. He has had a prominent part in directing the meteorological and hydraulic investigations, and the results set forth in detail in this paper will command general attention. Such records, covering one more of the greatest known floods of the Chagres, would have been of great service in the studies which determined the character of construction to be adopted—whether with lake and locks, or at sea level. The contours of the bed of the lake and the area of the watershed were then known only approximately, and the statistics as to rainfall and evaporation were much less complete than at present. It is pleasing to note that the new data confirm the conclusions and plan finally adopted, and set at rest any doubts as to the sufficiency of the water supply for all the operating needs of the Canal; and this without resort to the expedient of a reservoir above Alhajuela, to conserve the excess of flow in the rainy season which otherwise might go in part to waste over the spillway. Gen. Abbot.

It may be interesting to note that such a reservoir formed an essential element in the plan adopted by the New French Company, in order to augment the water supply afforded by the lake to be formed by the proposed dam at Bohio, a lake storing a much smaller volume than Lake Gatun, but which financial reasons compelled the Company to adopt, as the Old French Company had already excavated a sea-level

* This discussion (of the paper by Caleb Mills Saville, M. Am. Soc. C. E., published in January, 1913, *Proceedings*, and presented at the meeting of March 5th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Brigadier-General, U. S. A. (Retired).

Gen. Abbot. canal to the vicinity of Bohio. There is a favorable site for a masonry dam a short distance above Alhajuela, of which the cost was estimated at about $2 500 000, and Mr. Saville's surveys and observations in the Upper Chagres Basin have confirmed the opinion that a large volume of water may be stored there, if it is ever found to be desirable.

There are two reasons why such an addition to the present plan may be found to be advisable ultimately. The sudden and violent freshets which characterize the regimen of the Chagres will attain the canal route near Gamboa, where the narrow arm of Lake Gatun turns nearly at right angles to the direction followed by shipping. The channel here, for about four miles, is only 500 ft. wide, and it is not impossible that these sudden but brief rushes of water may prove to be more or less annoying. The largest freshets of record at Gamboa have carried maximum volumes ranging from about 65 000 to 79 000 cu. ft. per sec., with an average flow during 24 hours falling but little below 60 000 cu. ft. per sec. Their annual frequency, as well as the amount of rainfall on the Isthmus, seems to be subject to a progressive variation, ranging at Gamboa during the last 40 years from about 10 to about 35 annually, with a corresponding duration, including large and small freshets, ranging from about 75 to about 650 hours. A dam above Alhajuela would regulate this variable flow perfectly and eliminate all objectionable currents from the route for shipping. The second reason why an Alhajuela dam may become desirable is that, in these days of water-power development through the agency of electricity, it may be found convenient to augment the quantity of water available on the Isthmus by conserving the flow which otherwise, at such times, might run to waste over the spillway. Whether there will be such a demand can best be decided when the Canal is opened to traffic.

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INSTITUTED 1852.

PAPERS AND DISCUSSIONS

THE THEOREM OF THREE MOMENTS.
Discussion.

By Messrs. Milo S. Ketchum and J. P. J. Williams.*

Milo S. Ketchum, M. Am. Soc. C. E. (by letter).—The writer wishes to call attention to the "method of area moments" for the solution of statically indeterminate structures. This method, due to Mohr, is much easier to derive and shorter to apply to the calculation of constrained beams than either the "theorem of three moments" or the "method of least work." The author is in error in referring to the method of three moments as being an exact one. It is subject to the same limitations as the methods of least work and area moments, and all methods, if correctly applied, will give identical results. Mr. Ketchum.

Before applying the method of area moments to the problem of the plate-girder draw span with variable moment of inertia, it will be necessary to discuss briefly the method. The statement of the method of area moments is:

"If A and B are two points in a beam, the deflection of B with respect to a tangent at A, is equal to the moment about B of the area of the portion of the equilibrium polygon between A and B, multiplied by $\frac{H}{EI}$, where E is the modulus of elasticity of the material, I is the moment of inertia of the section of the beam, and H is the pole distance of the force polygon that was used in drawing the equilibrium polygon."

The proof of this statement, together with applications, has been given by the writer.† If the equilibrium polygon is drawn with a force polygon having a unit pole, the equilibrium polygon becomes the true bending moment polygon, and the statement of the principle is simplified accordingly.

* Author's closure.

† "Steel Mill Buildings," Chapter XIVa; and "Mine Structures," Chapter IV.

Mr. Ketchum.

The application of the method of area moments to a simple beam with a constant moment of inertia, loaded with a concentrated load and with a uniform load, respectively, is shown in the following problems:*

Simple Beam.—Concentrated Load at Center of Beam.—The simple beam in (*a*), Fig. 6, is loaded with a load, *P*, at the center. The bending moment diagram is shown in (*b*), and the beam is loaded with the bending moment diagram in (*c*), Fig. 6.

To find the equation of the elastic curve, take moments of the forces to the left of a point at a distance, *x*, from the left support, and the deflection of the beam at any point in the beam, (*a*), Fig. 6, will be equal to the bending moment at the corresponding point in the beam, (*b*), Fig. 6, divided by *EI*, and

$$-EIy = \frac{PL^2x}{16} - \frac{Px^3}{12} \qquad (1)$$

and

$$48\ EIy = P\ (4\ x^3 - 3\ L^2x) \qquad (2)$$

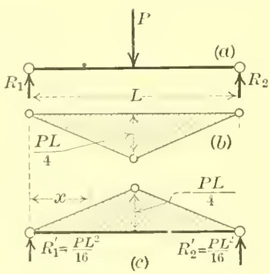

Fig. 6.

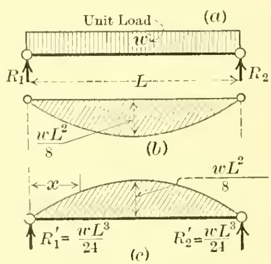

Fig. 7.

The maximum deflection will occur when $x = \frac{1}{2} L$, in Equation (2), or it may be found by taking moments of forces to the left of $x = \frac{1}{2} L$, to be

$$\Delta = \frac{PL^3}{48\ EI} \qquad (3)$$

Beam Uniformly Loaded.—The simple beam in (*a*), Fig. 7, is loaded with a uniform load of *w* per linear foot. The bending moment parabola is shown in (*b*), and the beam is loaded with the bending moment parabola in (*c*), Fig. 7. To find the equation of the elastic curve, take moments of forces to the left of a point at a distance, *x*, from the left support.

* Taken from "Steel Mill Buildings."

The equation of the bending moment parabola with the origin of co-ordinates at the left support is $y = \frac{1}{2} wLx - \frac{1}{2} wx^2$; the area of a segment of the parabola is Mr. Ketchum.

$$A = \frac{1}{4} wLx^2 - \frac{1}{6} wx^3 \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

and the center of gravity measured back from x is

$$-X_1 = \frac{x(2L - x)}{6L - 4x} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Taking moments of forces to the left of a point, x, and reducing, we have

$$24\,EIy = w(-x^4 + 2Lx^3 - L^3x) \ldots\ldots\ldots\ldots\ldots (6)$$

The deflection is a maximum when $x = \frac{1}{2} L$, and may be found directly by taking moments, or may be found from Equation (6), and is

$$\Delta = \frac{5\,wL^4}{384\,EI} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

For a beam with a variable moment of inertia, the transformed beams, (*b*), Fig. 6 or 7, should be loaded with the bending moment divided by the moment of inertia, *I*, at each point, or with the bending moment divided by *EI* at each point, if both the modulus of elasticity and the moment of inertia are variable.

The reaction of the center pier of a draw span of two equal spans with a constant moment of inertia and modulus of elasticity, may be calculated as follows:

Draw Span with Constant Section.—If the total span be $2\,l = L$, in Fig. 6, the deflection of the beam with span, L, loaded with a uniform load, w pounds per linear foot, is

$$\Delta = \frac{5\,wL^4}{384\,EI} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Now, if the center reaction, R_b, be considered as a load acting upward, it will be equal to a load, P, acting at the center of the span, which produces a deflection equal to the deflection of the simple beam, in Fig. 7, loaded with a load, w. The deflection of the beam loaded with a load, P, is

$$\Delta = \frac{1\,PL^3}{48\,EI} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Equating the values of Δ in Equations (7) and (8) gives the middle reaction

$$R_b = P = \frac{5}{8} wL \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Mr. Ketchum.

With the foregoing preliminary statement, we may now take up the solution of the problem of the plate-girder draw span with variable moment of inertia, which was solved by the author in Section 11.

Plate-Girder Draw Span.—The span $l_a = l_c = 60$ ft.; the total length is $l = 120$ ft. The girder is symmetrical about its center line. The relative values of the moments of inertia, I, at the following distances from the outside ends are:

0 to 12 ft., $I = 1$
12 to 36 ft., $I = 1.38$
36 to 48 ft., $I = 1.15$
48 to 60 ft., $I = 1.92$

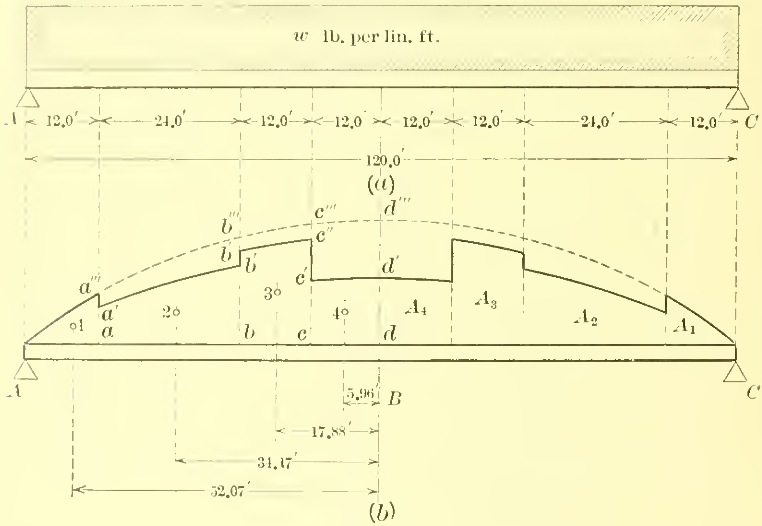

FIG. 8.

Solution.—Take a simple beam with a span of $l = 120$ ft., having the same cross-section as the draw span, and loaded with a uniform load of w pounds per linear foot, as in (*a*), Fig. 8. The bending moment diagram will be a parabola with a maximum ordinate at the center, $M = \frac{1}{8} w l^2 = 1\,800\, w$. To calculate the deflection in the beam due to the load, w, load the transformed beam in (*b*), Fig. 8, with $\frac{M}{I}$.

The areas of the segments of the parabola in (*b*), Fig. 7, were calculated by Equation (1), and are as follows:

$A_1 = 4\,032\ w$ (square-foot-pounds.)
$A_2 = 19\,618\ w$ " " "
$A_3 = 17\,030\ w$ " " "
$A_4 = 11\,100\ w$ " " "

Mr. Ketchum.

The centers of gravity of the segments of the parabola were calculated by Equation (4) as follows:

The center of gravity of A_1 about a is

$$x_1 = \frac{x(240 - x)}{720 - 4x} = \frac{12(240 - 12)}{720 - 48} = 4.07 \text{ ft.},$$

and the distance of the center of gravity of A_1 from the center of the beam, B, is 48 ft. + 4.07 ft. = 52.07 ft.

The center of gravity of the part of the dotted parabola about the point, b, is

$$x = \frac{x(240 - x)}{720 - 4x} = \frac{36(240 - 36)}{720 - 144} = 12.75 \text{ ft.}$$

The center of gravity of the dotted A'_2 about b will be

$$x_2 = \frac{(\text{Area } A_1 + \text{Area } A_2)\, 12.75 \text{ ft.} - \text{Area } A_1 \times 28.07}{\text{Area } A'_2}$$

$$= \frac{31\,104\, w \times 12.75 \text{ ft.} - 4\,032\, w \times 28.07}{27\,072\, w}$$

$$= 10.47 \text{ ft.}$$

and the distance of the center of gravity of A_1 from the center of the beam, B, is 48 ft. + 4.07 ft. = 52.07 ft.

The centers of gravity of A_3 and A_4 are calculated in the same manner.

The bending moment at the center of the beam, (b), Fig. 8, will be E times the deflection at the center of the beam, (a), Fig. 8.

$$M = E\,\Delta = R_1 \times 60 \text{ ft.} - A_1 \times 52.07 \text{ ft.} - A_2 \times 34.47 \text{ ft.}$$
$$- A_3 \times 17.88 \text{ ft.} - A_4 \times 5.96 \text{ ft.}$$
$$= 51\,780\, w \times 60 \text{ ft.} - 4\,032\, w \times 52.07 \text{ ft.} - 19\,618\, w \times 34.47 \text{ ft.}$$
$$- 17\,030\, w \times 17.88 \text{ ft.} - 11\,100\, w \times 5.96 \text{ ft.}$$
$$= 1\,850\,340\, w \text{ (cubic-foot-pounds)} \qquad (10)$$

Now load the same beam with a load of 1 lb., as in (a), Fig. 9. The bending moment diagram is a triangle with a center height of $M = \frac{Pl}{4} = 30$ ft-lb. The beam is loaded with the bending moment divided by the moment of inertia at each point.

The areas of the different sections of the moment diagram are:

$B_1 = 36$ (square-foot-pounds.)
$B_2 = 208.7$ " " "
$B_3 = 219.1$ " " "
$B_4 = 168.7$ " " "

The centers of gravity are as shown in (b), Fig. 9. Then the bending moment at the center of the beam (b), Fig. 9, will be E times the deflection of the beam (a), Fig. 9, due to a center load of 1 lb.

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$$M = E\,\Delta' = R_1 \times 60 \text{ ft.} - B_1 \times 52 \text{ ft.} - B_2 \times 34.00 \text{ ft.} - B_3 \times 17.71 \text{ ft.} - B_4 \times 5.78 \text{ ft.}$$
$$= 632.5 \times 60 \text{ ft.} - 36 \times 52 \text{ ft.} - 208.7 \times 34.00 \text{ ft.} - 219.1 \times 17.71 \text{ ft.} - 168.7 \times 5.78 \text{ ft.}$$
$$= 24\,137 \text{ (cubic-foot-pounds)} \quad \ldots\ldots (11)$$

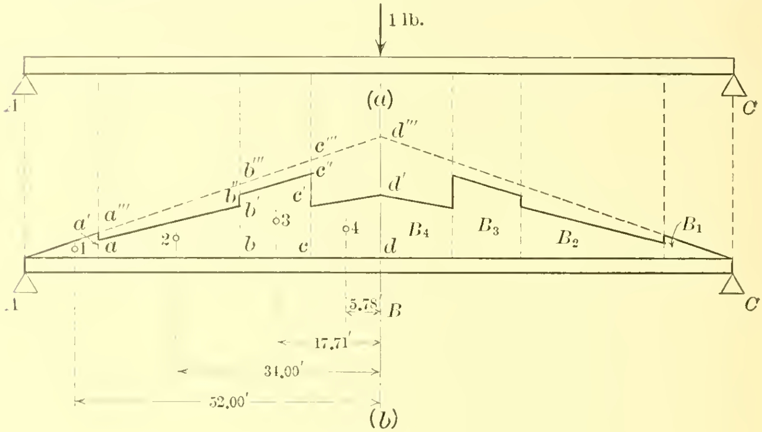

FIG. 9.

Now the center reaction of the draw span will be equal to Equation (10) divided by Equation (11), and

$$R_2 = 1\,850\,340\ w \div 24\,127\ w$$
$$= 76.67\ w.$$

The left reaction is

$$R_1 = \tfrac{1}{2}\ (120\ w - 76.67\ w)$$
$$= 21.665\ w.$$

The bending moment at the center support of the draw span is

$$M_b = 21.655\ w \times 60 \text{ ft.} - 60\ w \times 30 \text{ ft.}$$
$$= 500.1\ w \text{ (foot-pounds).}$$

This value checks the value calculated by the theorem of three moments.

The "method of least work" should give the same result as the "method of three moments" and the method of "area moments."

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J. P. J. WILLIAMS, ASSOC. M. AM. SOC. C. E. (by letter).—The discussion contributed by Mr. Ketchum, explaining and applying the method of "area moments," raises two questions of considerable importance. The first and most important is the question of the relative merits and ease of practical application of the three different methods which have been used in solving this particular problem of a continuous beam with variable moments of inertia. The second is the more general question of actual differences and relative exactness of these meth-

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ods, and also the broad question of methods of solution of statically indeterminate structures of any kind. The writer also intends to show the direct relation between the deflection formulas used analytically in the paper and the "area moment" method applied by Mr. Ketchum, and check some of the particular equations by the latter method.

In order to determine the relative amount of labor required to solve this problem, the writer has counted the number of "operations" required to make the solution for the center bending moment, M_b, by each of the three methods. By "operation" is meant, either a setting and reading of the slide-rule, or an addition or subtraction, also making allowance for cases of relatively easy additions or multiplications. The "area moment" method applied by Mr. Ketchum required 105 such operations, the tabular computation made by Mr. Turneaure, using a formula in terms of bending moments, required 78 operations, and the three moment method, using the writer's Equation (43) and Table 1, required only 41 operations. As the labor required to draw the diagrams for the "area moment" method is certainly as great as that of making the tables in the other methods, it is quite evident that, for this particular problem, the three moment method is far superior. It does not necessarily follow that, for other cases of different loading, or beams on more than three supports, the same method would still be best, but it is probable that it would, provided the mathematical integrations were handled rapidly.

The second question concerning the exactness of these three methods requires a clearer understanding of their essential differences. A careful reading of the first paragraph and the last three paragraphs of the paper will indicate that the term "exact method" used by the writer had reference only to the three moment method which included the effects of variable moments of inertia, settlement of supports, and slope of neutral curve. The principal object in view in Section 11 of the paper was the determination of the value of the error introduced in a particular case of a plate-girder draw span when the usual three moment formulas, neglecting effect of variable moments of inertia, were used; but, as Mr. Ketchum has raised the question of the relative exactness of the different "exact" methods, and has checked the writer's result so closely by using the "area moment" method, therefore indicating that the 2% discrepancy in Mr. Turneaure's result is probably inherent in that method, it is well to consider the different methods more fully.

Both Mr. Ketchum and Mr. Turneaure find the value of the center reaction by dividing the deflection of the girder assumed as simply supported at the ends (that is, with center reaction removed), and carrying the given load, by the deflection at the center caused by the unit load at that point. They differ only in the manner of computing these deflections, Mr. Ketchum using the "area moment" method and equations for

Mr. Williams. area and positions of centroids, and Mr. Turneaure using a tabular summation method, assuming the differential length, dx, in the equations to have a finite value of 6 ft. The formulas are essentially identical, but the exactness of the latter method depends directly on the length assumed for dx, which, theoretically, should be very small. The 2% discrepancy between the result of this method and the results of the other two is probably due to this fact. The three moment method which the writer used is also essentially a deflection method, as it was based on the General Equation (41) derived by Equation (8) for vertical relative deflections of the ends with respect to a tangent to the neutral curve at the center. Mathematically, therefore, all three methods are based on the same theory of deflection due to bending, and, except for the practical inaccuracies in application, should give the same results. The vital differences in the working equations used are only differences in details of application.

It should be emphasized, further, that all these methods give results which are inexact because of the neglect of shear deflection and of shear distortion of the assumed plane normal sections of the beam. Short, heavily-loaded girders with large concentrated loads are affected very materially by shear distortion, as has been shown in the writer's discussion* of "Faults in the Common Theory of Flexure"; and the common assumption of bending deflection only may be largely in error on the side of danger.

All statically indeterminate problems of any character can be solved by either of three general methods: (1) Method of Deflections or Deformations, (2) Method of Equal Work, and (3) Method of Least Work. All three methods give identical results when restricted to the same kinds of stress, but the first uses expressions for deflections or strains, the second equates internal work to external work, and the third applies the principle of minimum internal work.

For example, consider the very simple case of two elastic supports, a and b, subject only to the uniformly distributed direct stress caused by a load, W, carried in such a way that the total elastic strain of each support is the same.

Let W_a = portion of load, W, carried by a;

$W - W_a$ = portion of load, W, carried by b;

$e_a = \dfrac{E_a A_a}{L_a}$ = stiffness of support, a = total force required to produce total strain of unity in a;

$e_b = \dfrac{E_b A_b}{L_b}$ = stiffness of support b.

Then, the total deflection or strain $= \dfrac{\text{stress}}{\text{stiffness}}$.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 932-953.

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1.—*Method of Deflections.*—Equate deflection of a to deflection of b.

$$\frac{W_a}{e_a} = \frac{W - W_a}{e_b}; \text{ therefore } W_a = \frac{e_a}{e_a + e_b} W.$$

2.—*Method of Equal Work.*—Equate internal work to external work.

$$\frac{1}{2}\left[W_a \frac{W_a}{e_a} + (W - W_a)\frac{W - W_a}{e_b}\right] = \frac{1}{2} W \frac{W_a}{e_a}$$

$$\frac{W_a^2}{e_a} + \frac{(W - W_a)^2}{e_b} = \frac{W W_a}{e_a}$$

or $\frac{W_a}{e_a}(W - W_a) = \frac{(W - W_a)^2}{e_a}$; therefore $\frac{W_a}{e_a} = \frac{W - W_a}{e_b}$, as in 1.

3.—*Method of Least Work.*—Write the expression for internal work or resilience, R, and place the first derivative with respect to W_a equal to zero, to get condition for minimum.

$$R = \frac{1}{2}\left[\frac{W_a^2}{e_a} + \frac{(W - W_a)^2}{e_b}\right]$$

$$\frac{dR}{dW_a} = \frac{1}{2}\left[\frac{2\,W_a}{e_a} - \frac{2\,(W - W_a)}{e_b}\right] = 0; \text{ therefore}$$

$$\frac{W_a}{e_a} = \frac{W - W_a}{e_b}, \text{ as in 1.}$$

All three methods thus reduce to identical expressions, and it is possible to reduce even more complex problems, solved by the different methods, to the same working form. Usually, the method of deflection is the most direct and easiest of application. The vertical deflection of beams can be obtained in various ways: By the double integration of the fundamental equation, $M = EI\frac{d^2y}{dx^2}$, of the common theory of flexure; by the application of Equation (8) for $D_r = \sum_0^N \frac{Mn}{EI}x$; or by the semi-graphical method of area moments applied by Mr. Ketchum. They, also, are identical in result, if properly applied, and differ only in manner of procedure.

In order to show the relation between the last two methods, and also explain directly the "area moment" method in connection with that used in the paper, it is only necessary to note that $D_r = \sum_0^N \frac{Mn}{EI}x$ can be found by area moments. In values of $\frac{M}{EI}$ be plotted to scale as ordinates, and the lengths, n, be laid off on the x axis, then $\frac{Mn}{EI}$ will be an elementary area, and $\frac{Mn}{EI}x$ will be the moment of that area about the

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origin, O, the deflecting point. Then D_v = deflection of O with respect to a tangent to the neutral curve at $N = \sum_{O}^{N} \frac{Mn}{EI} x$ = total moment of area of diagram between limits O and N about O. Similarly, the change, A, in the angle between the tangents to the neutral curve at O and at N will be the area of the same diagram between limits O and N, because, by Equation (4), $A = \sum_{O}^{N} \frac{Mn}{EI} = \sum_{O}^{N}$ elementary areas = area between limits O and N. When E is constant, and M a continuous function of x, n becomes dx for a horizontal beam, and if A_i = area of a diagram plotted with $\frac{M}{I}$ as ordinates and x_i the distance of its centroid from the deflecting point as origin, then, for any given limits:

$$A = \frac{A_i}{E}, \text{ and } D_v = \frac{A_i\, x_i}{E}.$$

The last equation is the basis of the area moment method applied by Mr. Ketchum. When both E and I are constant, the ordinary bending moment diagram area can be used, because M is the ordinate and Mdx an elementary area.

Then
$$A = \frac{1}{EI}\int_{O}^{N} Mdx = \frac{A_m}{EI}$$

and
$$D_v = \frac{1}{EI}\int_{O}^{N} Mdx \,.\, x = \frac{A_m\, x_m}{EI}$$

in which A_m = area of bending moment diagram between limits O and N, and x_m = distance from centroid of A_m to origin O.

For example, the integration, $\int_0^{l_a} M\ dx\ x$ in Equation (30), can be more easily made by this method of area moments than by the analytic integration as given. Using areas of the bending moment diagram for M, Fig. 5, noting its component parts and that the origin is at A, the values of $A_m\, x_m$ will be:

For triangle due to M_a,

$$A_m\, x_m = \frac{M_a l_a}{2}\ \frac{l_a}{3} = \frac{M_a l^2_a}{6}$$

For triangle due to M_b,

$$A_m\, x_m = \frac{M_b\, l_a}{2}\ \frac{2\, l_a}{3} = \frac{2\, M_b\, l^2_a}{6}.$$

For triangle (not shown) due to bending moment caused by concentrated load, W_a, in simple beam, the area, A_m, would be:

$A_m = \frac{W_a\, z_a}{l_a}\ (l_a - z_a)\ \frac{l_a}{2} = \frac{W_a\, z_a}{2}\ (l_a - z_a)$ and distance from centroid to origin, A, is $x_m = \frac{l_a + z_a}{3}$.

Therefore, $A_m x_m = \frac{W_a z_a}{2} (l_a - z_a) \frac{l_a + z_a}{3} = \frac{W_a z_a}{6} (l^2_a - z^2_a)$, Mr. Williams.

as in Equation (31).

For the parabolic bending moment diagram (not shown) due to uniform load, w_a, on a simple beam, the maximum center ordinate being $\frac{w_a l_a^2}{8}$, the area $A_m = \frac{2}{3} \frac{w_a l_a^2}{8} l_a = \frac{w_a l_a^3}{12}$, and $x_m = \frac{l_a}{2}$.

Therefore, $$A_m x_m = \frac{w_a l_a^3}{12} \frac{l_a}{2} = \frac{w_a l_a^4}{24},$$

as in Equation (31).

The "equivalent beam" loaded with the bending moment diagram is a convenient method for finding, not only the deflection, y, but the slope, $\frac{dy}{dx}$, at any point of a given beam. As it will be of interest to note how this method is developed from the general equations already applied, consider the case of a simple beam, with constant E and I, and loaded uniformly with w pounds per foot, as in Fig. 2. The end reaction of the "equivalent beam" loaded with the parabolic bending moment diagram, as in Fig. 7 (c), is,

$$\frac{1}{2}\left[\frac{2}{3} \frac{wl^2}{8} l\right] = \frac{wl^3}{24} = \text{area between end and center.}$$

This area between end and center, however, when divided by EI, is the angle between the tangent at the end and the tangent at the center, when E and I are constant. As the tangent at the center is horizontal, the slope of the end tangent with the horizontal, therefore, is $\frac{wl^3}{24\,EI}$, as already obtained analytically in Equation (16). Thus slope of tangent at A equals reaction of "equivalent beam" divided by EI.

From Fig. 2, the slope at any point, X is:

$$\frac{dy}{dx} = (\text{slope of tangent at } A) - A_x$$

$$= \frac{\text{reaction of "equivalent beam"} - \text{area between } A \text{ and } X}{EI}$$

$$= \frac{\text{shear of "equivalent beam"}}{EI}.$$

Hence the slope, $\frac{dy}{dx}$, at any point, X, is equal to the shear at that point of the "equivalent beam" divided by EI. Using Equation (4) of Mr. Ketchum's discussion for the area of the segment of the parabola in Fig. 7 (c):

$$\text{Area between } A \text{ and } X = \frac{wlx^2}{2} - \frac{wx^3}{6} = \frac{wx^2}{24}(6\,l - 4\,x)$$

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Therefore
$$\frac{dy}{dx} = \frac{wl^3}{24\,EI} - \frac{wx^2}{24\,EI}(6\,l - 4\,x)$$
$$= -\frac{w}{24\,EI}[6\,lx^2 - 4\,x^3 - l^3],$$
which checks Equation (17), previously found in integration methods.

The deflection, y, at any point, X, Fig. 2, is seen to have the value:

y = (slope of tangent at A) x — deflection of X with respect to tangent at A

$$= \frac{\text{reaction of ``equivalent beam''}}{EI}\,x - \frac{\text{moment of area, } A \text{ to } X\text{, about } X}{EI}$$

$$= \frac{\text{bending moment of ``equivalent beam''}}{EI}.$$

Using x_1 = distance from centroid of area, A to X from the point X,
$$x_1 = \frac{x\,(2\,l - x)}{6\,l - 4x},$$
from Equation (5), given by Mr. Ketchum, there results:
$$y = \frac{wl^3}{24\,EI}\,x - \frac{wx^2}{24\,EI}\,(6\,l - 4\,x)\,\frac{x\,(2\,l - x)}{6\,l - 4\,x}$$
$$= \frac{-w}{24\,EI}[2\,lx^3 - x^4 - l^3x],$$
which checks the writer's Equation (18), or Equation (6), with opposite sign, as obtained by Mr. Ketchum.

This "equivalent beam" method of finding slope and deflection is perfectly general, and can be applied to beams with variable moments of inertia by a simple modification of the bending moment diagram, as already explained. The application made by Mr. Ketchum to the plate-girder draw span with variable moments of inertia, considered in the paper in Section 11, is a good illustration of this method of using an "equivalent beam," but, as already stated, it is much more laborious for this particular case than the three moment method giving identical results. Mr. Ketchum could have decreased by twelve the number of operations required for his solution by having used, instead of bending moment of "equivalent beam," simply the sum of the area moments about the end, as such sum gives the deflection of the end above the horizontal tangent at the center, which is equal to the center deflection desired for this particular symmetrical case.

MEMOIRS OF DECEASED MEMBERS.

NOTE. Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DANIEL SEYMOUR BRINSMADE, M. Am. Soc. C. E.*

DIED SEPTEMBER 7TH, 1912.

Daniel Seymour Brinsmade was born at Trumbull, Conn., on February 17th, 1845. He was the youngest son of Daniel Stiles Brinsmade and Catherine Mallette Brinsmade, his family having resided continually for more than 250 years within the limits of the original town of Stratford, Conn.

Mr. Brinsmade received his early education in the public schools of Trumbull and at the Gunnery at Washington, Conn. He then entered the Sheffield Scientific School of Yale University, from which he was graduated in 1870.

Immediately after his graduation, Mr. Brinsmade was appointed Assistant Engineer on the construction of the dam across the Housatonic River at Huntington, Conn., for the Ousatonic Water Company. In 1871, he was made Chief Engineer of the project, and afterward served successively as Secretary and Treasurer. In 1900, he was made President of the Company, which position he held until his death, at Shelton, Conn., after an illness of nearly two years. In 1891, the old dam was destroyed, and Mr. Brinsmade had to rebuild the structures and add many new features in order to meet the conditions which obtained at that time.

He made hydraulic engineering his life study, and, being an authority on the subject, was continually engaged in the construction of water supply and water power projects. He was widely known as a Consulting Engineer throughout New England and New York State; and served as a member of a commission appointed by the State of Maine to consider the question of water-power development in that State.

Mr. Brinsmade was a genial, cultured gentleman, whose advice was eagerly sought in business as well as in private life, and whose salient characteristics were honesty and truth. He took great pride in the betterment of his home town, Shelton, Conn., and was closely connected with all phases of activity within the community. He served the Borough in nearly every elective position in its gift. He was also elected to the General Assembly as Representative and State Senator, and, although a staunch Republican, he did his duty in these positions regardless of party.

* Memoir prepared by the Secretary from information on file at the Society House.

Mr. Brinsmade was a Director of the Shelton Water Company, a Trustee of the Griffen Hospital, President of the Shelton School Board for nearly twenty-eight years, President of the Plumb Memorial Library Directorate, and a Trustee of the Riverside Cemetery Association, all of which institutions he was largely interested in establishing and maintaining. He was also Vice-President of the Home Trust Company, of Derby, Conn., and a Director of the Birmingham National Bank. He was a Member of the Congregational Church, of Shelton, and also prominent as a Mason and an Odd Fellow.

Mr. Brinsmade was elected a Member of the American Society of Civil Engineers on February 1st, 1888. He was also a Charter Member of the Connecticut Society of Civil Engineers and served as its President in 1907.

HENRY FISHER WHITE, M. Am. Soc. C. E.*

DIED OCTOBER 28TH, 1912.

Henry Fisher White, son of J. Avery and Jane Elizabeth (Fisher) White, was born at Boston, Mass., on October 14th, 1851.

On the completion of his education, Mr. White, in 1870, entered the office of Messrs. Buttrick and Wheeler, at Worcester, Mass., which firm was engaged in general city and county practice. He also began his career as a railroad engineer about this time as Transitman on the Fitchburg Railroad.

In 1872, Mr. White was appointed Assistant to the Division Engineer of the Cairo and Vincennes Railroad on the construction of tunnels and heavy trestles on that road. He remained in this position until March, 1873, when he was employed as Transitman on a preliminary survey for a local company between Brookfield and Kansas City, Mo.

In June, 1873, Mr. White accepted the position of Division Engineer on the Ware Branch of the Boston and Albany Railroad, in charge of track-laying and ballasting on old grades. In 1874 and 1875, he was again engaged on general engineering work for Messrs. Buttrick and Wheeler, and on surveys for, and the construction of, minor roads. In September, 1875, he was employed as Transitman on location, and, in February, 1876, as Resident Engineer in charge of construction, of the Dayton and Southeastern Railway in Ohio.

In June, 1876, Mr. White was appointed Assistant to the Chief Engineer of the Burlington, Cedar Rapids and Northern Railway, with headquarters at Cedar Rapids, Iowa. In November, 1881, he was made Chief Engineer, which position he held until the road was made part of the Rock Island System. During his incumbency as Chief Engi-

* Memoir prepared by the Secretary from information on file at the Society House.

neer, the Burlington, Cedar Rapids and Northern Railway grew from 700 to 1 300 miles of splendidly built and equipped road.

When, in 1902, this road was absorbed by and became part of the Rock Island System, Mr. White was appointed Engineer of Maintenance of Way of the latter company, with headquarters at Chicago, Ill. He remained in this position until February, 1908, when he retired from active work. His later years were spent between his home in Chicago and his ranch near Montrose, Colo., where he died from an attack of heart failure.

On April 14th, 1880, Mr. White was married to Miss Anna McConnell, of Cedar Rapids, Iowa, who, with two daughters, survives him.

Mr. White was elected a Member of the American Society of Civil Engineers on January 2d, 1890. He was also a Member of the Western Society of Engineers and of the American Railway Engineering Association.

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